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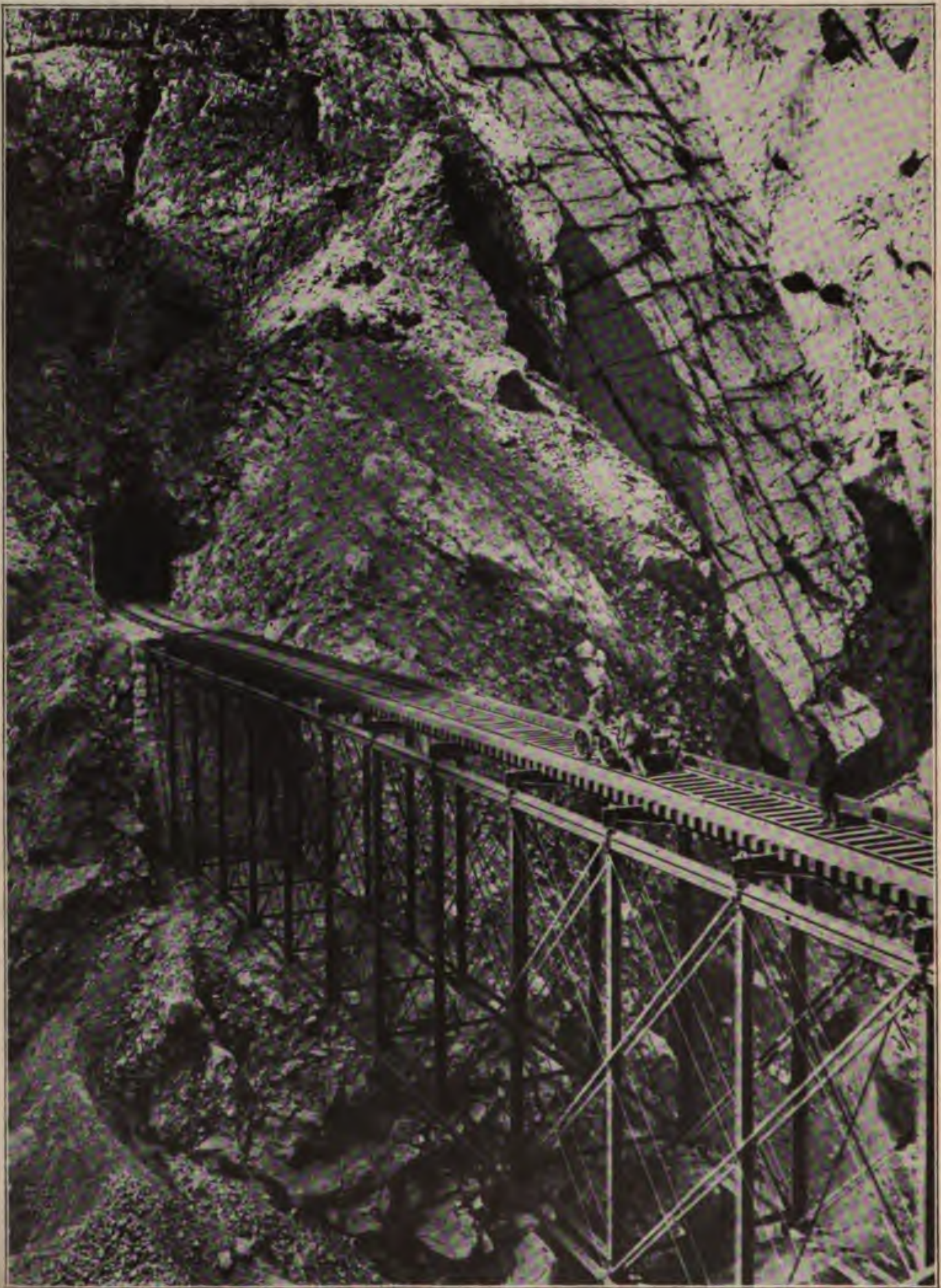

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Cyclopedia
of
Civil Engineering

A General Reference Work

ON SURVEYING, RAILROAD ENGINEERING, STRUCTURAL ENGINEERING, ROOFS
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CONSTRUCTION, HYDRAULIC ENGINEERING, IRRIGATION,
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EIGHT VOLUMES

CHICAGO
AMERICAN SCHOOL OF CORRESPONDENCE
1908

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Grateful acknowledgment is here made also for the invaluable co-operation of the foremost Civil, Structural, Railroad, Hydraulic, and Sanitary Engineers in making these volumes thoroughly representative of the very best and latest practice in every branch of the broad field of Civil Engineering; also for the valuable drawings and data, illustrations, suggestions, criticisms, and other courtesies.

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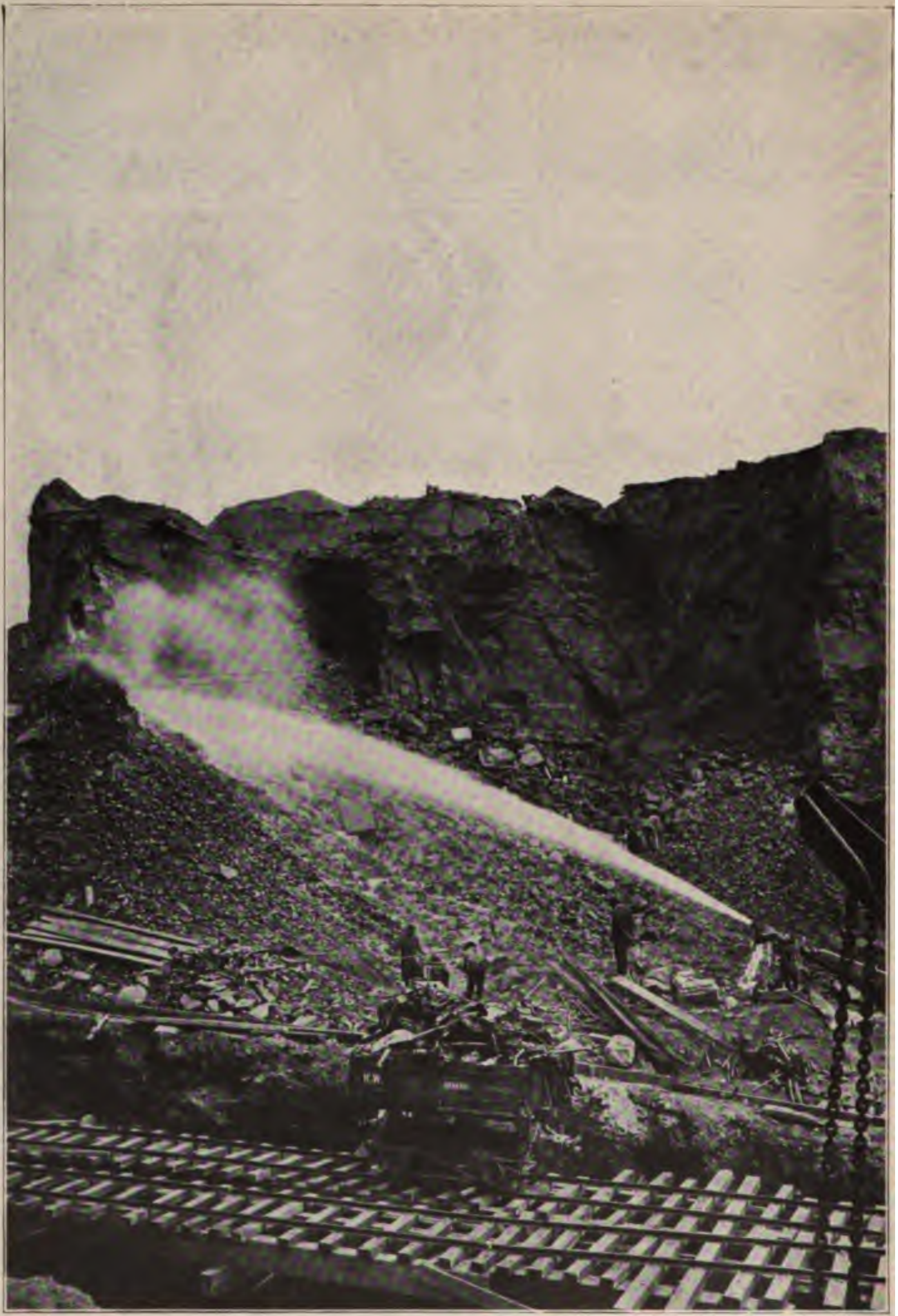

HYDRAULICKING IN RAILROAD CONSTRUCTION

View near Seattle, Washington. Tearing down a hill by hydraulic power, and sluicing earth through a flume.

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Foreword

THE marvelous developments of the present day in the field of Civil Engineering, as seen in the extension of railroad lines, the improvement of highways and waterways, the increasing application of steel and reinforced concrete to construction work, the development of water power and irrigation projects, etc., have created a distinct necessity for an authoritative work of general reference embodying the results and methods of the latest engineering achievement. The Cyclopedia of Civil Engineering is designed to fill this acknowledged need.

¶ The aim of the publishers has been to create a work which, while adequate to meet all demands of the technically trained expert, will appeal equally to the self-taught practical man, who, as a result of the unavoidable conditions of his environment, may be denied the advantages of training at a resident technical school. The Cyclopedia covers not only the fundamentals that underlie all civil engineering, but their application to all types of engineering problems; and, by placing the reader in direct contact with the experience of teachers fresh from practical work, furnishes him that adjustment to advanced modern needs and conditions which is a necessity even to the technical graduate.

¶ The Cyclopedia of Civil Engineering is a compilation of representative Instruction Books of the American School of Correspondence, and is based upon the method which this school has developed and effectively used for many years in teaching the principles and practice of engineering in its different branches. The success attained by this institution as a factor in the machinery of modern technical education is in itself the best possible guarantee for the present work.

¶ Therefore, while these volumes are a marked innovation in technical literature — representing, as they do, the best ideas and methods of a large number of *different* authors, each an acknowledged authority in his work — they are by no means an experiment, but are in fact based on what long experience has demonstrated to be the best method yet devised for the education of the busy workingman. They have been prepared only after the most careful study of modern needs as developed under conditions of actual practice at engineering headquarters and in the field.

¶ Grateful acknowledgment is due the corps of authors and collaborators — engineers of wide practical experience, and teachers of well-recognized ability — without whose co-operation this work would have been impossible.

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U. S. ARMY MAP MAKERS AT WORK IN THE FIELD

PLOTTING AND TOPOGRAPHY.

DEFINITIONS AND OUTLINE OF WORK.

1. **Definition.** Topography is the art of making a graphic representation, (a) of the outlines of all water courses and bodies of water, (b) of all forms of relief of the surface of the ground, and (c) of all artificial features and structures built by man. In general the art consists in representing all these features in such a way that they are most easily intelligible from the map; while economy requires that the methods employed shall be such as to make a map which shall give all the necessary information with a minimum of work in surveying and drafting.

2. **Scale of Map.** (A general discussion.) One of the first things to be decided is the scale of map to be used, for on this decision will depend the character of the work which is to be done. For example, if the map is to be plotted on a small scale it will be useless to do the surveying work with a closeness of detail which it will be impracticable to represent on such a small scale. The scale of map must depend on the use which is to be made of it, as will be discussed later. On the one hand a very small scale will mean that desired details cannot be readily shown or else can only be shown by the draftsman using the most extreme care, and on the other hand, a large scale will mean that if the survey covers a considerable area of ground the map will be inconveniently large. In brief, the true criterion for the scale of the map is that it should be drawn to the smallest scale which will properly represent the topographical features which must be accurately shown on the required map.

3. **Method of Field Work.** (Determination of its general character.) The choice in the method of field work depends on the same considerations as govern the required scale of map. On the one hand, the method adopted should have a *practicable* accuracy sufficient to properly represent all required features. The practicable accuracy of any given method is only known to those who have learned by actual experience the uncertainties involved in that method. But on the other hand, time and effort should not be

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sacrificed in plotting minute details with painstaking accuracy when it is known by experience that such accurate work cannot have any practical utilization. When an engineer is making an "exploratory" survey through a comparatively unknown tract of country, he may be content with determining elevations with a barometer, the directions of the lines with a hand compass, the slope of lines with a hand level or clinometer and distances with a pedometer (pace-measurer) or odometer (an instrument which records the revolutions of a wheel of known circumference, which is rolled over the route traversed). These crude methods have the merit of great rapidity and cheapness and can be made with sufficient accuracy to determine whether that general route should be immediately rejected or deserves further consideration and a more accurate survey. The above, which is called a *reconnoissance*, applies chiefly to railroad work. When a topographical survey of a large area is to be made regardless of the method of utilization, such a reconnoissance survey is made chiefly for the purpose of planning the detailed work, selecting triangulation stations (if the triangulation method is to be used) and determining the method best suited for the results to be accomplished. For example, military surveys are frequently made on horseback or from a boat passing on a stream or lake. The three elements of distance, direction and elevation are determined with only a low degree of accuracy, but in the hands of trained men it is sufficient for the purpose of these surveys. To pass to the other extreme, geodetic surveys, which have as their ultimate object the determination of the form of the earth, employ the most refined methods which ingenuity can devise to eliminate sources of inaccuracy which even in ordinary surveying are considered negligible.

4. **Plotting.** (The general method to be adopted.) This, similarly, depends on the method of field work and the scale of the map. Small scale maps are used to represent a very large area of country. Usually nothing is indicated except the most important features. The outlines of the largest bodies of water, such as oceans, lakes and rivers are represented as closely as is practicable. Important cities are represented by arbitrary signs such as dots and small circles. For special purposes (such as for railroad maps) the railroad is represented in a very exaggerated way. But on the other hand, as the scale of the map grows larger, more and more of the

details can be represented, until on the larger scale maps, even the nature of the crops in individual fields will be represented by appropriate topographical signs. When the relief of the ground is to be shown, and especially if it is of importance to represent it with accuracy, the usual method is to show it by contours. Since these lines are apt to be mistaken for other lines, such as roads, rivers, etc., the device of employing different colors for the various topographical signs is employed. When one individual map is to be made, it is practically as easy to use the various colors as to employ the single color of black, and it is therefore usually wise to employ different colors for such maps. But when maps are to be printed the color method will require several impressions for each sheet, which is expensive. This feature will be discussed later. If it were practicable to take a photograph of the entire area of country to be surveyed from a balloon, which was at a sufficient height above the ground to be surveyed, it would give what might be considered in some respects an ideal representation of the area to be surveyed. But such a picture would be in many respects useless. Many small but most important features would be almost indistinguishable. The essence of proper plotting consists in representing, perhaps in an exaggerated form, those features which it is especially desired to show on the map, and omitting other details which are of no practical importance, at least for the particular purpose for which that map is made. The method of plotting contours also permits an accurate representation of differences of elevation which would be unobtainable from a balloon picture.

Since the final result of a topographic survey is the map and since the method of surveying to be used depends on the scale of the map and the nature of the physical characteristics which are to be represented on the map, the various methods of mapping will be first described.

TOPOGRAPHIC MAPPING.

5. **Large Scale Maps.** Topographic maps are frequently made as the basis on which to design constructive work, such as buildings, factories, dams, reservoirs, sewerage and waterworks systems, canals, railroads, etc. Each case has its own peculiar features with regard to the topographical details which must be

represented and the accuracy with which they must be measured and plotted, but a scale of 100 feet per inch is usually large enough, unless the details are to be given with great precision or the whole area is so small that a scale of 50 or even 10 feet per inch will not make a large map. At a scale of 100 feet per inch, every building can be drawn in its actual form and dimensions (to the nearest foot), and even the separate rails of a railroad track may be clearly indicated. At such a scale, a map 30 inches square (including the border) will show about a half-mile each way or a quarter of a square mile in area. This is sufficient for many purposes and the scale is therefore largely used. There are but few natural objects (such as are usually required to be shown on maps) which cannot be shown in their natural scale. This, however, does not apply to the conventional signs used to indicate various kinds of vegetation—when it is required to show these. An accurate photographic view of these things would be useless as well as impracticable, and a conventional sign for each, which is frequently a suggestive indication of each on an exaggerated scale, must be employed even on the largest scale maps.

6. Small Scale Maps. Even when large scale maps are separately drawn to indicate details at special places, there is a great advantage in having maps on a scale so small that a very large area may be shown on a single map of convenient size. On such maps, although the general locations of buildings, etc., are desired, their exact shape or size is comparatively of no importance. The United States Geological Survey indicates residences even on maps of the scale of approximately one mile to the inch by minute squares of solid black. They are but little over $\frac{1}{100}$ of an inch square, and, therefore, would indicate (strictly) a square building a little over 50 feet square. Even a far greater approximation as to size and shape would serve the purpose. As the area to be represented grows larger this requirement must be met by (a) increasing the size of the map until it approaches the limit of convenient size for handling, (b) by making sectional maps, which for some uses are impracticable, or (c) by decreasing the scale. When the scale is decreased the same data *may* be shown by plotting the work finer. This has been carried to an extravagant limit by the United States Coast and Geodetic Survey, whose maps are exceedingly expensive to prepare, require the finest grade of draftsmen, and which almost require a microscope to determine the details. Perhaps the most practicable method of decreasing the scale is by judiciously eliminating details which will have little or no value for the immediate purpose for which the map is made, or by modifying the conventional signs employed in such a way that they will convey sufficient information without sacrificing simplicity and clearness. As we decrease the amount of matter placed on a map, we increase the clearness and ease of distinguishing those things which are shown. The United States Coast Survey maps, being made for general use, are so crowded with minute details that they have a very "flat" appearance, each line being necessarily so very fine that it requires very close examination to use the map, and this renders it difficult to use the map in a "broad" way, with the eye at a distance from it. While the inexperienced engineer should be very cautious to avoid the omission of details which might prove useful or essential and should err on the side of surveying and recording too much *in the field*, yet the experienced engineer will save time in the field and improve the character of his map by omitting details which his experience tells him will certainly not be needed in the work for which the map is to be used. When an engineer is making a map for a special purpose, such as railroad work, he has no justification in crowding the map with details which might be of importance in some other kind of work, but which will not under any circumstances be of importance in the contemplated railroad construction.

7. Contours. A contour is the intersection of the irregular surface of the earth by a level surface. The student should note that, although a level surface is practically a plane surface when small areas are considered, yet it is really a curved surface, being the surface that the ocean would assume if it were to rise to the assumed level. The simplest method of appreciating the nature and location of contours is to consider them as the shore lines formed by the ocean if its surface were to be raised successively to the different levels assumed for each contour. The various levels are always taken at equal vertical intervals, the intervals varying with the scale of the map and the character of the country. For the closest of detail work on a larger scale, such as might be used for landscape gardening, a contour interval of one or two feet might be necessary. But for ordinary purposes five feet or more is used as the contour interval. The topography required for preliminary railroad surveys is usually done with a 5-foot contour interval. The topographical surveys as made by the United States Geological Survey, and which are plotted at a scale of one or two miles per inch (approximately), are made with a contour interval varying from 20 to 100 feet, the 100-foot interval being necessary where steep mountain ranges are to be represented.

There are certain principles regarding contours which the student must keep in mind and by which he may avoid errors in doing this work. Barring a few exceptional cases which will be mentioned later the following principles may be laid down: First, a contour is always a continuous line inclosing an area; second, since that area may, and frequently does, run off the map the contour may run from one edge of the map to any other point on the edge; third, a contour line is always a continuous line—it never stops abruptly; fourth, contour lines do not cross each other or merge into each other. The exceptions to the above rules occur only where the contours run into an artificial vertical wall or a precipice which is actually vertical. In the very unusual case of an overhanging cliff the contours might actually cross each other very slightly, but with such exceptions which would be readily recognized when they occur, the above principles may be considered as true, and when the principles are violated on a map, it may be considered as evidence of incorrect work. Neatness of work requires that the contours should be inked in by lines of uniform color and width. In order to make sure that the lines are of uniform width they should be inked in with a "ruling pen" and never with a "nib pen." If the inking in of the contours is done in one continuous operation, the lines will probably have a more uniform color and thickness. The contour elevations are always given with reference to some datum plane, such as sea level, and the elevations are always a multiple of the contour interval. For example, if the contour interval is 20 feet and the datum plane is sea level, then each contour elevation will be a multiple of twenty, even though the whole tract is far above sea level. The contours might be at elevations 660, 680, 700, 740, etc. Then the 600, 700, 800-foot contours would be made extra heavy, and could be more readily distinguished even in steep places where the contours would be very close together.

The contours should be numbered at frequent intervals, so that the elevation of a contour at any point may be determined by following it for a very short distance to the nearest "ladder" of numbers. The student is referred to Plates I, II and III for illustrations of this point as well as many others.

Hachures. The representation of the relief of a country by hachures has been almost entirely superseded by the above described method of contours. Hachures are objectionable, first, because their representation with even a pretence of accuracy and neatness is exceedingly laborious; secondly, because at their very best they do not have the accuracy of contours; and thirdly, the map is so covered by them that other desired topographical forms are practically crowded out. The chief use of such a method is to give a general idea of the character of the country when the survey has not been made in sufficient detail to render possible an accurate representation of contours. Under such conditions there is usually such a lack of detail on the map that the hachures will not crowd out the other desired topographical features. Incidentally the hachures when well done will make a very "pretty" map, when the map is really lacking in topographical detail. Maps of exploratory surveys are apt to have this character. The hachures can be used to give a very approximate representation of mountain peaks, gorges, etc., whose forms are determined only by very approximate sketching when the observer is possibly one or two miles away from them, and absolutely no instrumental work is resorted to in determining their elevation. In order to construct hachures with an approximation of accuracy it is desirable to sketch in contour lines with a pencil at equal vertical intervals. The hachures are lines which are assumed to be perpendicular to a contour line at every point of their length, or in other words, and speaking geometrically, they represent in each case and at every point of their length the steepest line at that point of the surface. On this principle the lines should be perpendicular to the contour lines and also perpendicular to any intermediate contour which might be interpolated between the contours which are drawn. This means that the hachures should be in general more or less curved. (The student should study Fig. 1 while reading this explanation.) The steepness of the slope will therefore be indicated by the length of the separate hachure lines, but the steepness is also indicated still more graphically by increasing the thickness of the hachures on the steeper slopes. In other words, hachures are made shórter and wider for steep slopes and longer and narrower for flatter slopes. The method of hachuring has been developed so as to attempt to indicate the degree of slope by a systematic variation of the distance between the lines, but this method is exceedingly laborious and very inaccurate unless an amount of time is spent on it, which is very wasteful and impracticable. The general principle in hachure representation is that the darker the shading the steeper is the slope, and that any considerable space in white means a surface which is practically level. In Plate IV is shown a representation of the hachure method as employed on a large scale by the United States Coast Survey. The student may at once observe the exceedingly laborious character of the work and the fact that the hachures cover up a space which might otherwise be utilized in the representation of other topographical features. Hachures have the advantage of calling attention to the steepness of slopes, which may be of extreme importance on military maps where accuracy is by no means essential, but on which the fact of a steep slope, even if it is no more than a bank of earth, is of importance. Hachures may thus be used to represent an artificial or natural embankment when there is no other representation on the map showing any relief features of the ground.

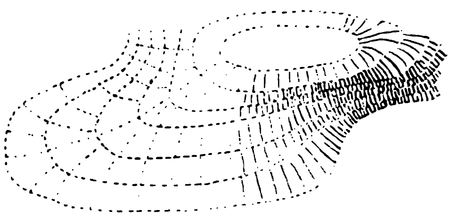


Fig. 1. Hachures (Dotted Lines Indicate Pencil Construction Lines which are Eventually Erased.)

Plate V illustrates the Coast Survey method of representing relief by means of contours and yet employ but one color. A comparison of Plate V with Plate III shows at once the great value of using three colors to represent the various topographical features. It also shows the practical difficulty even by the competent draftsmen employed by the Coast Survey to make such a completed map in but one color. Although the contours are undoubtedly more accurate and will give a close approximation of the elevation of any desired point, yet the hachured map of Plate IV is actually the more pleasing.

8. Determination of Contours. The method of determination of contours depends on the scale of the map and of the accuracy with which the work must be done. Several methods are here described which vary in accuracy, in the instruments employed and in the labor required for doing the work.

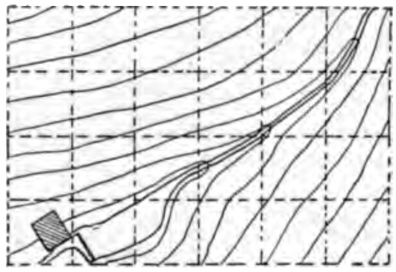


Fig. 2. Contours—Gridiron Method.

"Gridiron" method. This simple method consists in establishing a "gridiron" of squares or rectangles over the desired area and determining the elevation of each intersection point. The great advantage of this method is its simplicity. The disadvantages are that neither the contours nor the salient points will in general agree with the points determined. Many points are observed needlessly while other essential points located within the rectangles are only determined by approximation from adjacent points. The work may be done approximately with hand level, rod and tape, but a transit and wye-level are also used with this method.

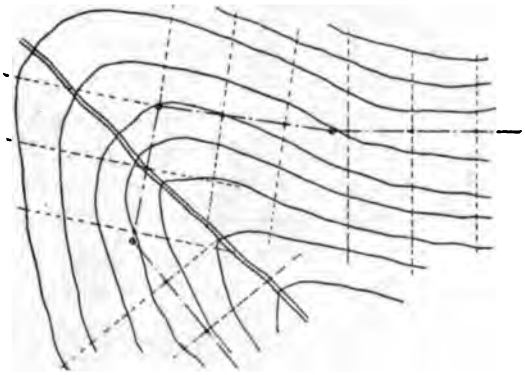


Fig. 3. Contours—Cross-Section Method.

"Cross-section" method. A broken line, consisting of straight lines between instrument stations is run with a transit or other angle measurer and a tape. At certain intervals, usually 100 feet, cross lines are run at right angles to the traverse lines. See Fig. 3. The elevation of the 100-foot sub-stations are all determined, and from

them the elevation and position of points on the offset lines become known. Usually the intersection of the offset lines with the various contours is determined. The method of doing this is illustrated in Figs. 4 and 5. Knowing the elevation of the sub-stations, a point on the offset line is readily found by trial which is at the required contour elevation. The location of points with vertical intervals of 5 feet is then very simple, as shown in the figure. The contours are thus determined directly. The above methods are only used when the areas are comparatively small (but perhaps of indefinite length, as in railroad work), or when the contour interval is small and close results are essential.

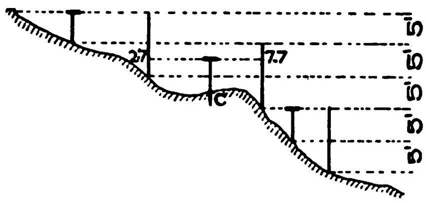


Fig. 4. Locating Contours.

"*Stadia*" *method.* This method has a skeleton like the traverse method, but even the distances between stations and their elevations are obtained by the stadia system. No 100-foot sub-stations are located. From each station point, "shots" are taken to all desired points, not only for the sake of establishing contours but also to locate all desired topographical features. Incidentally the elevations of these points, which would be determined, will assist in determining the contours more closely. It is generally found in practice that when all desired details have been located in position and elevation, the contours are determined with but few, if any, additions of points which are located solely for contour purposes. The mathematical theory of the stadia has already been given in Plane Surveying, and therefore will not be repeated here. Only practice will teach the engineer how to locate the stadia points so as to obtain all the necessary information and also to obtain the most information with the least surveying.

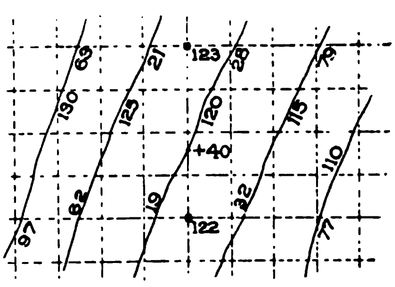


Fig. 5. Contours—in Note Book.

9. Plotting the Contours. As before stated, the points are supposed to be so located that straight lines joining any two adjacent points will lie approximately in the surface of the ground. On this assumption the location of any contour which comes between two located points is a mere matter of dividing the horizontal distance between the points into parts which are proportional to the differences of elevation of the given points and the contour. For example, the elevation of one point is 162.0 and of another 167.8. The 165-foot contour will evidently lie between them and at $\frac{165.0-162.0}{167.8-162.0}=\frac{3.0}{5.8}$ of the horizontal distance between them from the lower point. This is the theoretically exact solution. But since it depends on an approximation the experienced draftsman can make a rapid mental solution of the proportion which will be as accurate as the data justifies. If there are two or more intermediate contours the same general method may be applied for each, but the simplest method will be to locate first the contours immediately adjacent to each point and then interpolate any intermediate contours. After determining where each contour will cross the line joining each pair of surveyed points the contours may be drawn in by connecting the corresponding intersections. Practical modifications of this theoretical method will be made by the topographer or draftsman to allow for minute variations in the contours which may be sketched in with sufficient accuracy since the area between determined points will always be small.

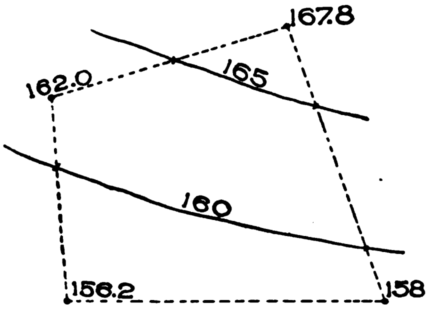


Fig. 6. Plotting Contours.

Practice among topographers varies very greatly in the details of obtaining contours, varying from the one extreme of making the method as purely mechanical as possible (which implies excessive work if it is at all accurate) to the other extreme in which instruments are only used to obtain a few points of control and then to fill in the intermediate spaces by sketching. Men of long experience will do marvellously close work with very few and very simple instruments, but the inexperienced man will do more reliable work by obtaining with close instrumental accuracy all of the important points and contour points with close frequency, only depending on sketching to fill in very small areas in which inaccuracies would be unable to accumulate.

10. Telemetry. This name is applied to the various methods of measuring distances without the direct use of a tape or chain. No space will here be given to a description of the many ingenious but essentially impracticable special instruments which have been devised for this purpose. In general they are incapable, for practical reasons, of giving accurate results, or else they are expensive to construct, heavy to carry, comparatively useless for other purposes, and therefore not generally adopted. The three methods hereinafter described have the advantages that they are merely attachments to an engineer's transit and that the additions to the weight and cost of the instrument are comparatively insignificant.

11. Vertical Arc Method. From one standpoint, this method hardly deserves to be classed as a regular method. Its disadvantages are so great that no one should depend on it as a standard method, but it is frequently useful when no other method is available. The essential principle is that of the solution of a triangle, of which one side and two angles are known. The rod is supposed to be held vertically. In the special and most simple case shown in Fig. 7, a rod reading is taken with the line of sight level. Then a rod reading is taken with the line of sight inclined at some measured angle. In the right-angle triangle thus formed the horizontal distance equals the rod reading divided by the tangent of the vertical angle. But in general the rod will be at some elevation with respect to the transit so that neither line of sight is level. In Fig. 8 the angles a and b are both measured. The angle $c = 90° - a$. Then in the triangle $I\ J\ H$ two angles and a side are known and the two other sides may be computed, and from them and the vertical angles the true horizontal distance and the difference of elevation may be computed. The one advantage of this method is that it can be used with any transit, which is provided with a vertical arc. The disadvantages are many; (a) two vertical angles must be measured and recorded for each point

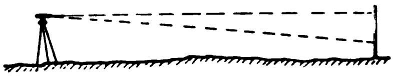


Fig. 7. Vertical Arc Method.

observed; this adds to the field work; (b) an oblique triangle and then a right triangle (or two right angle triangles, as illustrated) must be solved for each point observed; this makes the office work very laborious, and it is impracticable as a regular method, in spite of the fact that it may be systematized by tabulating the results for even values of the vertical angle, the most convenient even values being used and the corresponding rod readings observed; (c) the practical accuracy obtainable is vitiated by the fact that the transit is handled and the telescope is moved in the interval of taking the two readings; if the base of the instrument were mathematically immovable during the complete operation (as it is assumed to be) there would be no trouble, but it is practically known that a microscopically minute jarring may take place which may be enough to vitiate the accuracy of the work done; (d) at a distance of 500 feet, and a rod reading intercept of 5 feet, the vertical angle would be 0° 34′ (to the nearest minute); the vertical arcs of most transits will not read closer than minutes and many of them are not dependable even for that; in the above case, an uncertainty of one-half minute in the vertical angle would mean an uncertainty of over seven feet in the distance, which demonstrates the practical limit of accuracy of the method.

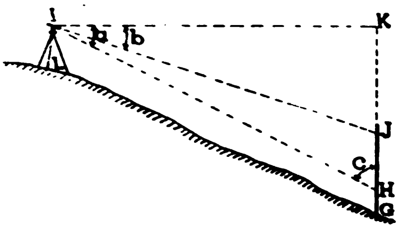


Fig. 8. Vertical Arc Method—Sloping Ground.

Example. In Fig. 8 let the angle $a = 5° \, 30'$, the rod reading being 1.72, and the angle $b = 4° \, 50'$, the rod reading being 6.45.

From the figure we may write

$$I\,K = \frac{K\,J}{\tan 4° \, 50'} = \frac{K\,J + J\,H}{\tan 5° \, 30'}$$

$$\therefore K\,J \tan 5° \, 30' = K\,J \tan 4° \, 50' + J\,H \tan 4° \, 50'$$

$$K\,J\,(\tan 5° \, 30' - \tan 4° \, 50') = J\,H \tan 4° \, 50'$$

$$K\,J = \frac{J\,H \tan 4° \, 50'}{(\tan 5° \, 30' - \tan 4° \, 50')} \qquad J\,H = 6.45 - 1.72$$

$$\therefore I\,K = \frac{K\,J}{\tan 4° \, 50'} = \frac{6.45 - 1.72}{(\tan 5° \, 30' - \tan 4° \, 50')} = \frac{4.73}{.01173} = 403 \text{ feet}$$

$$K\,J = 403 \times \tan 4° \, 50' = 34.08. \quad K\,G = 34.08 + 6.45 = 40.53.$$

If the height of the instrument, $I\ L$, = 4.90, then the difference of elevation of L and G is 40.53 - 4.90 = 35.63. As an illustration of the probable inaccuracy of this method, a variation of one-half minute in the value of either of the vertical angles will make a difference of over five feet in the computed value of $I\ K$.

12. Gradienter Method. The instruments for gradienter work are shown in Figs. 70, 71 and 75, in Part II. of Plane Surveying. The fundamental principle and use of the gradienter is described in the first part of Part III of Plane Surveying, and it therefore will not be repeated here. The method of using the gradienter to obtain distances and differences of elevation when the line of sight is inclined, are also given, and it is shown that the reduction equations are exactly the same as those required for stadia work. The practical difference between gradienter and stadia work, therefore, lies in the relative rapidity of the mechanical work and in the relative accuracy obtainable. It should be noted that the gradienter requires two settings of the horizontal cross-wire on the rod. This is sometimes done by setting the screw of the gradienter at certain even divisions, so that it is turned, say 2, 3 or 4 whole revolutions between the two sights and then reading the rod at whatever reading there may happen to be; or the two readings on the rod are taken at even-foot divisions on which the cross-wire is more easily set, and then the screw readings are read at whatever point they may happen to be. The latter method is probably the more accurate and rapid, especially if two targets placed on a level rod have been used to sight at. For all inclined sights the vertical angle must also be read. The time required is greater than that for stadia work, but the worst feature of the gradienter method is the limitation of practical accuracy. The theory presupposes that the base of the instrument is absolutely rigid during the complete operation of taking the upper and lower readings, but the telescope must be moved, the instrument must be touched, and the gradienter screw turned between the taking of the two readings. The mere fact of touching it and the interval of time required between the two readings is often sufficient to cause a motion of the instrument, which, although microscopically minute, will be sufficient to vitiate the accuracy of the work done, and unfortunately there is no practical means of detecting whether any such inaccuracy has actually occurred. In stadia work no motion of the instrument is made between taking the upper and lower readings. The instrument may (and should) be absolutely untouched. The eye can examine the reading for both upper and lower wire repeatedly to see whether any motion has occurred in the very short interval of reading the separate wires. In the case of the gradienter, the motion may or may not take place. Even if no inaccuracy were caused in this way, even once in a hundred times, there is always the uncertainty and impossibility of detecting such an error unless the readings are repeated, which, of course, would be a great waste of time. The method of reduction of stadia observations by means of tables, diagrams, or logarithmic slide rule, which is elaborated further on, may be applied equally well to the reduction of gradienter observations.

The disadvantages of the gradienter may be summed up as follows: (a) The screw thread wears rapidly until it becomes inaccurate; (b) it requires considerable time to take each observation as it should be taken; (c) the possible jarring of the instrument while any one set of observations are being taken makes the results uncertain if not actually inaccurate. The chief advantage lies in the facility with which lines of any desired grade may be run. After being properly levelled, the number of turns and fraction of a turn indicates immediately the grade of the line of collimation of the telescope in *per cent.*

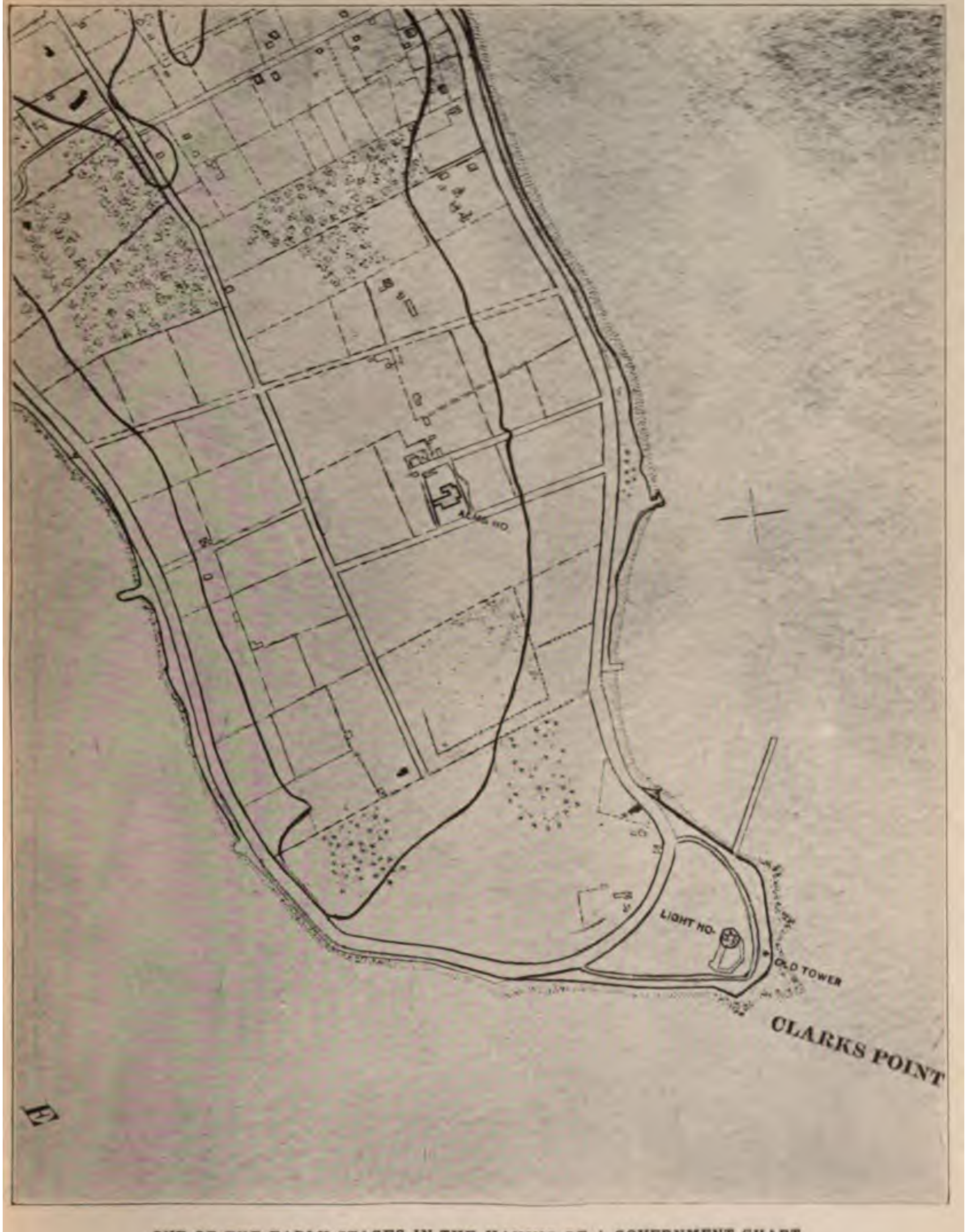
13. Stadia Method. The theory of the stadia has already been given in Plane Surveying (pp. 120–125), and therefore, will not be repeated here. The methods of making and recording the field observations were also elaborated. The work of "reducing" the observations is, however, very laborious, unless it is facilitated by some of the methods hereafter described. In general every observation is made at some angle above or below the horizontal. Theoretically this reduces the nominal horizontal distance somewhat and requires a computation for the difference of elevation. Practically, the correction for the horizontal distance is too small for notice when the vertical angle is very small, but even the limitations of that correction must be known by computation. Restating the equations given on page 125 of Plane Surveying we have

$$\text{Hor. Dist. } C'N = \frac{f}{i} s' \cos^2 I + (f + c) \cos I. \qquad \text{and}$$

$$\text{Diff. Elev. } BN = \tfrac{1}{2} \frac{f}{i} s' \sin 2I + (f + c) \sin I.$$

In these equations s' is the rod intercept, f the focal length, i the distance between the stadia wires in the telescope and I the angle of inclination of the line of sight. $\frac{f}{i}$ is usually made 100.

14. Numerical Accuracy Obtainable in Stadia Measurements. Of course, a vertical arc is an essential feature of a transit used for stadia work, but the accuracy of the vertical angles depends on the accuracy with which the whole instrument is levelled. The plate bubbles of an ordinary engineer's transit, while amply accurate for their primary use of making the plates so nearly horizontal that the accuracy of horizontal angles is not vitiated, seldom have a sufficient accuracy to obtain vertical angles as close as the nearest minute of arc, and frequently their accuracy is far less than this. One method of obviating this inaccuracy is to determine by repeated reversions of the instrument, while it is being set up, that the horizontal plate is truly horizontal, or in other words, that the vertical axis is truly vertical, thus eliminating any error of adjustment of the plate bubbles. But even this method, unless repeated every few minutes at a costly sacrifice of time, will not prevent inaccuracies due to the settling of the instrument; nor will it at its best render possible any close accuracy of the vertical angles, since the bubbles are not sufficiently sensitive. A transit for stadia work usually has a large bubble underneath the telescope, which is much more sensitive, and on which a movement of 30″ of arc will correspond to about one division. Even if this bubble is in perfect adjustment, it will be necessary to make a test every few moments by clamping the telescope at exact horizontality and noting whether the vernier of the vertical arc reads zero or reads the amount of its "index error." When it does not do so, it indicates that the instrument has settled somewhat in the ground and has thereby become thrown out of level to some extent. A far better method is to have the vernier of the vertical arc readily adjustable and with a sufficiently sensitive bubble attached to it. When this bubble has been properly adjusted, any slight motion of the instrument due to settlement can be readily neutralized by an adjustment of the vernier arc, which will then give each vertical angle correctly. But it should be noted that since vertical angles are only read to single minutes, the sights should not be made excessively long except when they are taken merely for somewhat approximate pur-



ONE OF THE EARLY STAGES IN THE MAKING OF A GOVERNMENT CHART

Photographed from the original plane-table sheet on which results of survey were recorded in the field. For later stages in the completion of this chart, see pages 42 and 58.

poses in which a considerable inaccuracy would be tolerable. For example, at a distance of 1000 feet a difference in vertical angles of 0° 1′ will equal .29 of a foot or 3½ inches. Such a stadia shot would be justifiable at that distance to obtain a "contour point," but unless the whole work was approximate, a stadia station should not be located with such inaccuracy. On the other hand, for short distances the stadia method gives results which are amply accurate for its purpose. It is not, of course, supposed that the method would be used to replace wye leveling. But when, as is usually the case, the stadia method is used for preliminary surveys or else to fill in the topography between the points whose relative positions and elevations have already been determined by triangulation and by leveling, then errors have no chance to accumulate and the method has the combined advantages of great rapidity, cheapness and sufficient accuracy for the purpose. One great bar.to its use has been a lack of knowledge regarding the principles involved or a lack of knowledge of the fact that the reductions can be so readily made by means of slide rules. Much of the work now being done is "reduced" by the use of tables and numerical multiplications, which are certainly very slow and laborious and which would almost condemn the method for practical use if they could not be avoided. In making the above reductions, the student can solve them by the use of trigonometrical tables, although that is the most laborious method. A set of stadia tables (see Table I) is given with this course of instruction, and the student should solve the problems given later on, using these tables; if in addition he can secure the use of a stadia slide rule, he can determine for himself the economy of time obtainable. Of course the student must expect that it will be a little difficult at first to use any stadia slide rule until he becomes accustomed to its manipulation, but a short experience with it will convince him that it will pay for itself in a very few days. If possible, the student should solve the problems (as well as others which he can himself make up) by all of the three methods referred to, and thus determine for himself the relative facility of doing the work.

OFFICE WORK.

15. Reduction of Observations. The ordinary numerical trigonometrical reduction of each observation by the equations of § 13 is so laborious that such a practice would render the method impracticable; $(f + c)$ is usually about 12 to 15 inches; $\cos I$ is always less than unity, but for ordinary small angles is but little less than one; therefore $(f + c) \cos I$ is always very nearly one foot. Considering that one foot is the smallest unit of measurement which it is practicable to use in observing stadia *distances*, the practical accuracy is not vitiated by considering that term invariably as one foot. When the angle I is about 5° 45′, $\cos^2 I = .99$ and the horizontal distance would be 1 per cent less than its nominal value of 100 s', or 100 times the rod reading. At one-half this angle (2° 52′) the correction would be about $\frac{1}{4}$ of 1 per cent. This gives an indication of when the correction will be large enough to deserve accurate computation. It may be also observed that for small angles $\sin I$ is nearly equal to $\frac{1}{2} \sin 2I$; for 10° the discrepancy is about $\frac{1}{4}$ of 1 per cent; therefore we may say approximately that the difference of elevation $BN = (100\,s' + 1)(\frac{1}{2} \sin 2I)$. For angles of less than 10°, such as would usually be the case between stadia stations when the levels are particularly important, the error is a very small fraction of an inch and unobservable with the instrument. Even with a vertical angle of 30°, the error of the approximation would be less than 0.1 foot, which is the lowest unit for contour levels. We may therefore deduce the following practical method in which all approximations are within the lowest units of measurements. For small vertical angles, the horizontal distance equals 100 times the rod reading plus one foot. For larger vertical angles multiply 100 times the rod reading by the square of the cosine of the angle of inclination and add one foot. The practical method of doing this will be given later. For the difference of elevation, add one foot to 100 times the rod reading and multiply this by one-half the sine of twice the angle. There are three shortened methods of accomplishing these trigonometrical multiplications; (a) *tables*, (b) *diagrams*, and (c) *slide rules*.

The first is by means of tables which shorten the operation to the extent of giving directly $\cos^2 I$ and $\frac{1}{2} \sin 2I$ for all desired values of I, but this involves two numerical multiplications which are tedious. See Table I.

The second method is by means of a large chart or diagram. There are many forms of these, but they are usually essentially as follows: there are two sets of ordinates, one representing distance (the nominal rod reading times 100) and the other the desired quantity. Lines are drawn from the origin for each desired angle so that the intersection of any angle lines with any distance ordinate will give an ordinate of the other system which will give the desired quantity. The practical difficulty lies in the fact that if made conveniently small, the scale is so small that the results are inaccurate. When they are sufficiently enlarged they are clumsy to handle. The larger sizes should be used and they are capable of good work.

The third method is by means of slide rules. These are but an adaptation of the ordinary slide rule since the process is merely one of multiplication. One scale is marked with degree and minute marks at the proper places for the numerical values of those trigonometrical functions. By setting the zero of the angle scale at the given distance (100 times the rod reading) on the other scale and then noting the mark for the observed vertical angle, it will be found opposite the required difference of elevation. The true horizontal distance may be most easily found by modifying the above formula so as to compute the *correction* to the horizontal. $\text{Cos}^2 = 1 - \sin^2$. Therefore by multiplying 100 times the rod reading by $\sin^2 I$, we obtain a correction, which is seldom more than 5 or 10 feet, which is subtracted from the nominal distance. In the practical use of the scale it will be found that this quantity may be more readily read off from the scale than the true horizontal distance, and as it is only a few feet the subtraction is readily made mentally while it is being recorded. Colby's slide rule only gives the difference of elevation. The K. & E. stadia slide rules are engine-divided on white facings and give both the difference of elevation and true horizontal distance with one setting of the rule, but the length of the logarithmic unit is very short, which makes it correspondingly difficult to obtain close values. The Webb stadia slide rule has a logarithmic unit 12.5 inches long, which was made possible by the use of the cylindrical principle, and this enables the values to be easily read with as close accuracy as stadia work will justify. The horizontal distance is obtained by computing the correction to the horizontal from the *sin*^{2} function.

16. Numerical Examples. 1. The rod reading for a stadia shot was 3.74 and the vertical angle was + 6° 22′. What is the true horizontal distance and the difference of elevation? The horizontal distance would be, according to the approximate rule, $100\,s' \cos^2 I$. Cos 6° 22′ = .9938. Squaring this number we have .98765, which, multiplied by 374 = 369.4. Adding 1 foot we have 370.4. The only approximation in the above is on the assumption that $(f + c) \cos I = 1$ foot, but as before stated, the error of the approximation is so small that it may be disregarded. In fact, the tenths in the distance given above should also be disregarded, since the rod reading is taken only to the nearest hundredth of a foot, which corresponds to an even foot in horizontal distance. The difference of elevation is evidently equal to $\frac{1}{2} \times 100 \times 3.74 \times \sin (2 \times 6° 22') + (f + c)$ sin 6° 22′. This reduces to 41.22 + 0.11 = 41.33. Since sin 6° 22′ = .1109 while $\frac{1}{2}$ sin 12° 44′ = .1102, the approximation referred to above only equals .0007 (f + c) which equals in this case .0084 inches when the $(f + c) = 12$ inches. Since it is useless to attempt to take levels closer than 0.01 of a foot which equals .12 of an inch it is thus seen that the above approximation is far within the limits of accuracy. Therefore we can see at once that the difference of elevation equals $(374 + 1)\,\frac{1}{2} \sin 2I$, which in this case equals 41.33, which is the same value as before.

The above solution was made by the long and usually impracticable process of taking the multipliers from an ordinary trigonometrical table. The method is simplified by employing what is called a "stadia reduction table," such as is given in the back of this book. Looking in the column headed 6° we find opposite 6° 22′ the value 98.77. If we multiply 98.77 by 3.74, the rod reading, we obtain 369.4 as before; adding 1 we have 370.4. For the difference of elevation we multiply the number corresponding to 6° 22′, which is 11.02, by 3.74, and obtain 41.21. This practically is the same value as before, the difference, which is really but a fraction of a hundredth of a foot, being caused by a slight inaccuracy in the last figure in the table. At the bottom of the table we find that when $(f + c) = 1$, as we have assumed above, the quantity to be added for difference of elevation = .11, which gives a total of 41.32, which is the value we previously obtained by the long process. This process is therefore considerably shorter than the first, although it involves two multiplications. The third method is to use a diagram. As it is impracticable to illustrate a diagram which would have any practical value for this purpose on the pages of such a book, we must be content with a mere description of the process as described in section 15. Stadia slide rules are illustrated later on. (See Section 33, Figs. 25, 26 and 27.) To use them we simply need to set the zero of the angle scale (or perhaps some other specified mark in a multiple scale) opposite the mark on the distance scale which represents the rod reading; then opposite the mark on the angle scale, which indicates the angle of elevation or depression, we may read directly on the distance scale the difference of elevation. For example, in the above case, and using the Webb Stadia Slide Rule, we would set the left-hand end of the scale, which gives the "difference of elevation" and which contains the angles between 6° and 7°, on the mark indicating 375, then looking for 6° 22′ on the angle scale, which would be found by interpolating between the mark for 6° 20′ and 6° 25″, we may read at once 41.3. This is always sufficiently accurate for contour work, and unless extreme care has been taken to have the instrument in perfect adjustment it is as close as the difference of elevation can be depended upon at a distance of 375 feet. This operation requires but a few seconds of time. To obtain the correction to the horizontal distance, we place the left-hand end of the rule giving "Hor. corr." for 6° 22′ on 375 of the main scale and note that the angle marked 6° 22′ occurs at $\frac{1}{100}$ of 460, showing that the horizontal correction is 4.6, which would give the true horizontal distance as 370.4 as before. This is even more accurate than is necessary, and we would call the true horizontal distance the even number 370 feet.

Example 2. Verify the following results obtained with an instrument whose $(f + c) = 12$ inches. It should be noted that a large vertical angle is not usually combined with a great distance, as that would mean an extreme difference of elevation. The following examples have purposely been chosen to cover almost every case that will occur in actual practice; namely, short distances and slight difference of elevation; long distances with small vertical angle; and short distances and large vertical angles, such as occur when surveying through a gorge and side shots are taken up the steep banks of the gorge. The student should note in each case that the approximate rules will usually apply.

Rod Reading.	Vert. Angle.	Diff. Elev.	True Hor. Dist.
1.34	+ 0° 14′	+ .05	135′
8.76	+ 0° 12′	+ 3.06	877′
4.22	- 7° 06′	- 51.9	417′
0.96	+ 25° 15′	+ 37.2	80′

17. Reduction of Field Notes. This topic is mentioned chiefly with a view of giving a warning. With a few exceptions which will be noted, there should be no such thing as a "reduction of field notes," or in other words, the notes taken in the field should be complete and the surveyor should never wait to get back to the office and then depend on his memory to complete the notes. A desire to make the notes neat and to do the work under more favorable circumstances of office surroundings and suitable temperature, will often tempt the surveyor to make the field notes very meagre and then depend on them and on his memory to make a more complete set after the office is reached. Under special circumstances this may be justifiable, but in general every essential fact should be recorded in the field. The only exception to this may be stated in a broad way to be such reductions and additions as can be unmistakably made from the field notes taken. An example of such reduction is the computation of the difference of elevation and the correction to the horizontal distance which must be computed from stadia measurements. When circumstances will permit, it is preferable to make even these reductions in the field, as may be readily done if a stadia slide rule is used, and then plot the work on a sketch board. The surveyor should cultivate the habit and ability to make sketches in the field which shall be approximately to a correct scale and which will settle indisputably the ambiguity which often arises from field notes expressed simply in words and numbers. Therefore with the exception of mathematical processes, which are perfectly definite and which may be made more conveniently in the office after the surveying work is done, there should be no "reduction of field notes."

18. Plotting of Farm Surveys from Field Notes. Some of the statements in the last paragraph apply especially to this subject. A sketch of the farm, as is illustrated in "Plane Surveying," page 119, Fig. 83, should always be made in the note book. Such a sketch is of even greater importance than the more formal notes in columns such as are illustrated on page 118, for the sketch alone would serve the purpose if care is taken to record everything on it, while the notes in columns are frequently subject to ambiguity. To put it more strongly, it requires the greatest skill to make the formal notes so that they will not be ambiguous. The plotting of farm notes will therefore consist of making a copy of the sketch, but with all dimensions drawn accurately to the desired scale instead of roughly and inaccurately as in the sketch. The scale of sketch to be adopted of course depends on the amount of detail which it is desired to show on the map. If possible, the map should be oriented so that the meridian points up, but this may be impracticable on account of the form of the farm, or the direction of its longest dimension. As a matter of practical accuracy, the direction of the meridian line should be immediately determined by penciling a long line that runs approximately through the center of the map, then all lines should be plotted in accordance with their angle from this meridian line. If latitudes and departures have already been computed it is far more accurate to determine the direction of the boundary lines by scaling off the latitude and departure of each line. If it is necessary to use a protractor, then the protractor should be as accurate as possible. The very small protractors having a radius of about 2 inches, which are frequently found with a cheap set of drawing instruments, are worthless for accurate topographical plotting. If a protractor must be used, it is far better to purchase an 8-inch or even a 14-inch paper protractor which can be bought for 20 or 40 cents. With care this will last a long time and will be far more satisfactory than the small metal or horn protractors which are frequently used.

Another method of laying off angles accurately is to use a table of natural tangents, using a decimal scale, say 20 to the inch. Five inches of this scale is divided into 100 parts and by interpolation it may be read to 1000 parts. Lay off this 5 inches on a line and erect a perpendicular to that line at the 5-inch point. Suppose that the angle to be laid off is 16° 45′; the natural tangent taken from a table is 0.301. Therefore, scale off on the perpendicular with the "20 scale" 30.1 divisions. A line to the starting point will make the required angle of 16° 45′ with the given line. Since the length of the perpendicular would be unreasonably long with angles which approached 90°, the preferable method would be to erect a perpendicular to the given line at the point from which the angle was to be made; lay off the 5 inches on that perpendicular, erect a line perpendicular to it (or parallel to the original line) and determine the tangent of 90° minus the given angle. Of course, if still greater accuracy is desired, a longer base line than 5 inches can be used. Other details of the plotting of farm surveys, which are applicable to other forms of plotting, will be given subsequently.

19. Plotting of Profile From Level Notes. Profiles are almost invariably plotted on "profile paper." There are three general methods of ruling the lines on profile paper, and the paper adopted will depend somewhat on the use to be made of it. Vertical lines are ruled $\frac{1}{4}$ inch apart, sometimes $\frac{1}{6}$ inch apart. Horizontal lines are ruled 20, 25 or 30 to the inch. The paper comes either in sheets about 42 inches long and 15 inches high, or else in rolls which are 10 or 20 inches wide and 50 yards long. Sometimes the paper is mounted on muslin and sometimes the rulings are made on tracing paper or on tracing cloth. The levels are always referred to some datum plane, which is lower than any elevation to be recorded. Each fifth horizontal line is ruled somewhat heavier than the others and one of these lines should be selected as the line indicating some even multiple of 5 feet above datum plane. Some little ingenuity is often necessary to choose the proper line, so that the profile will run neither too high nor too low. The scales, both horizontal and vertical, which are to be used, must be selected with reference to the character of the work to be shown. The horizontal scale will usually vary from 100 to 400 feet to the inch. At a scale of 400 feet to the inch, each vertical line therefore corresponds to 100 feet of horizontal dis-

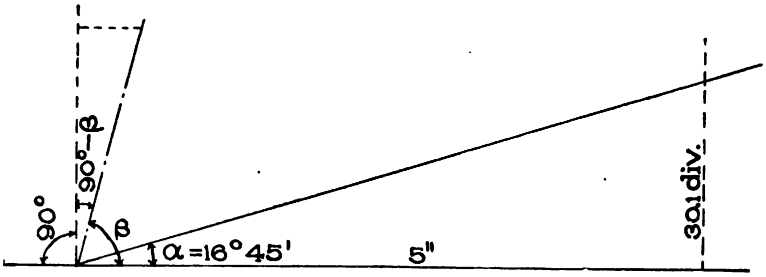


Fig. 9. Plotting Angles from Tangent.

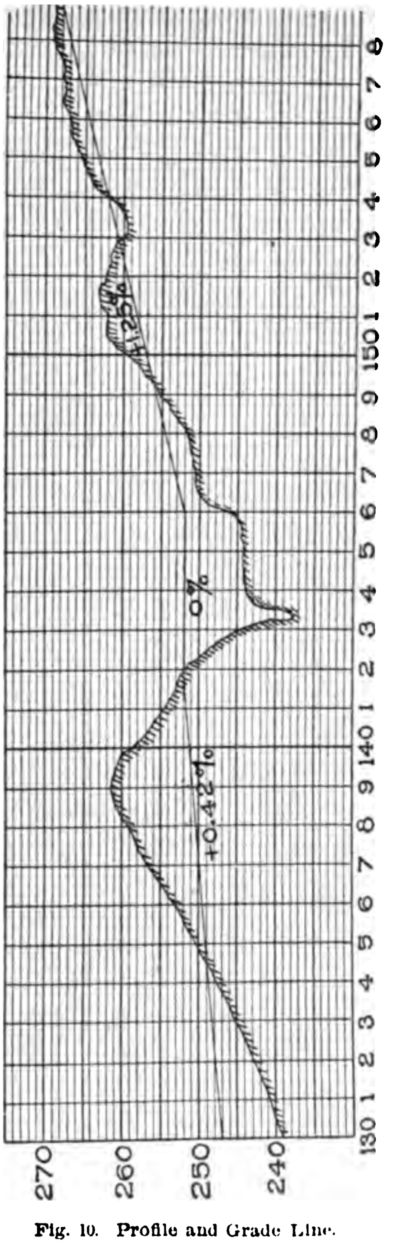


Fig. 10. Profile and Grade Line.

tance. Usually each horizontal line (spaces $\frac{1}{20}$ of an inch apart) means 1 foot of elevation, or in other words, the vertical scale is 20 feet to the inch. Since the engraved lines usually cover a vertical space of 15 inches, this scale will allow a range of profile of 300 feet. If 20 feet per inch is chosen as the vertical scale, and the total vertical range of the profile does not exceed 300 feet, then the whole profile may be placed without a break on such a sheet, but the choice of lines to represent any given elevation must be made accordingly. When a larger vertical scale is chosen or the range of elevation is greater, as it is apt to be for a continuous railroad profile, a break may be made by raising or lowering the profile an even number of feet; for example, by making a change at some vertical line by which the elevation indicated by a given horizontal line is abruptly changed, say 100 or 200 feet.

Mark on the profile sheet the elevation corresponding to the horizontal lines and also the distances corresponding with the vertical lines, then having recorded in the note-book the elevations above the datum plane of every desired point in the profile, plot each point at its proper position according to the horizontal scale and likewise according to the vertical scale. A line drawn through these several points will give the profile of the surface. If it is desired to establish a given grade line on that profile, the location in distance and elevation of the governing points of that grade line should then be plotted and the grade line drawn through them. Usually this grade line will lie alternately above and below the surface line and we thus have a graphical representation of the amount of cut or fill required at any point.

20. Plotting of Stadia Survey Notes. Stadia surveying is almost invariably based on a skeleton which has been surveyed with a higher grade of accuracy than is possible with mere stadia work. This skeleton is usually made by triangulation, and therefore errors cannot accumulate beyond those incident to a survey between two triangulation stations. Of course, the sub-stations between the triangulation stations are determined wholly by stadia measurements, and if the area to be surveyed is very small, this may constitute the skeleton of the whole work regardless of any triangulation stations. If triangulation has been used, then the triangulation stations should be first plotted with all the accuracy that is possible. The next step will be to run the traverse lines which connect the stadia sub-stations and which run between the triangulation stations, as well as shorter traverse lines which might be used to connect stadia stations and form a net work. The stadia work for the lines between stations should be done with such accuracy that there is no appreciable error or closure when connecting the triangulation stations. If any such error is found, it should be investigated, and if it amounts to a great deal, it may indicate some gross blunder in the field work. One method of investigating such blunders is to compute the latitude and departure of each course of the traverse. The algebraic sum of these should be the latitude and departure between the two terminal stations, and the amount of the discrepancy will indicate whether it should be ascribed to mere cumulative inaccuracy or to some gross blunder at some one place. When the closure has finally been settled with satisfactory accuracy all stadia stations will have been plotted. The reference line for azimuths in stadia work is usually a meridian line, but whether it is so or not, that reference line should be drawn through each station and sub-station from which stadia shots have been taken. A full circle protractor is the most convenient instrument to use for this purpose, and although a protractor 6 inches in diameter, when used carefully, may answer the purpose, it is preferable to have one of a larger size even though it is engraved on paper. Number all the stadia shots at each station and with its zero line running along the azimuth reference line, mark by dots on the edge of the protractor the angles as given for the various shots taken from that station. As each one is marked, indicate the mark very lightly by a number, which is the number of the shot. Usually all the shots taken from any one station may be marked off with a single placing of the protractor; then, removing the protractor, scale off from the stadia station the various distances for each shot, making a dot at the required distance and drawing around it a small circle very lightly in pencil to aid in finding its position. Indicate immediately in pencil the elevation of the point, which elevation should have been previously computed and recorded in the note book. This also serves to identify the point. When two or more points have been obtained to determine the corners of buildings, bridges or other structures, the points should be immediately connected so as to render the significance of the points at once more intelligible. It may even be preferable to indicate very lightly in pencil the general run of the contours through the elevations which have been obtained, since every new indication on the map will render it plainer and enable the rest of the work to be put on with less danger of making blunders. By making all marks with a hard pencil and without bearing on very hard, mistakes may be corrected without excessive erasures, which might spoil the paper, and yet the penciling will be sufficiently plain so that it can be inked in when it is found to be satisfactory. This is particularly true in the matter of connecting the contour lines which have been determined from various stadia stations. After indicating all desired features, the azimuth reference line through each stadia sub-station, as well as other features which have been lightly penciled in, may be erased without trouble.

21. Plotting of Maps by Projection. For small areas such as farms and parks, where the area is but a few square miles, it is sufficiently accurate to assume that the surface of the earth is plane, or in other words, that all plumb lines held at all points of the map would be precisely parallel. But when the areas amount to hundreds of square miles, and particularly when they cover whole continents, the curvature of the earth cannot be neglected either in the computations or plotting. A "single curved surface," such as that of a cone or cylinder, can be "developed" and it will lie flat; *i.e.*, the surface could be rolled out on a plane surface without changing any proportions. But a surface of "double curvature" such as a sphere is not developable, and there then arises the problem of representing on a plane surface the surface of a sphere with as little distortion as is possible. There are many methods of accomplishing this, some of which are very approximate. They will be described in the order of their complexity, which likewise means the order of their accuracy. In each case will be described the method of plotting parallels of latitude and meridians of longitude. Since in any case a line is determined by its passing through a point of known position and running at a given angle with the meridian, the plotting of meridians and parallels gives a sufficient framework for the plotting of any map.

Trapezoidal Projection. In this method parallels and meridians are plotted as straight lines. In Table II are shown the distances between parallels of latitude at any given latitude (the distance varying slightly with the latitude) and also the distance between meridians on any parallel of latitude. A straight vertical line at the center may indicate the even degree meridian which passes approximately through the center of the map to be plotted. On this line lay off distances at the proper scale which will represent the distances between the parallels of latitude included within the map. On two of these parallels, which are approximately at one-fourth the distance from the top and bottom, lay off distances from the central meridian, which represent, according to Table II, the positions of even degree meridians at so many degrees each side of the center. Connecting the corresponding points on these two parallels we have a series of trapezoids, each of which have parallel lines for top and bottom and a pair of converging lines for the sides. When the scale is such that the angular distance of any point from the center of the map is but a few degrees measured in latitude or longitude, then this method might be tolerated, although the errors at the outer edges of the map would be considerable. When the map includes many degrees of latitude and longitude, then the errors become intolerable.

Simple Conic Projection. A cone is readily developable and its development is a sector of a circle whose radius is the slant height of the cone. If we consider a cone to envelop a sphere so that its vertex is in the pole of the sphere and is tangent to the sphere at some parallel of latitude, which is purposely chosen as the middle parallel of the desired map, then if points on the surface of the sphere are projected on to the surface of the cone and the cone is then developed, we will have a representation of the surface of the earth with comparatively little distortion, especially when the angular distance measured in degrees of latitude from the middle parallel is very small. Fig. 12 illustrates this method on a very small scale, but it also shows that when the angular range of the map is very large the distortion on the outer edges of the map is very considerable. The distance between parallels 45° and 60° is very perceptibly less than that between 0° and 15°. A very obvious modification of the method is to space the parallels at their true distance apart. To apply this method lay off as before a vertical line which represents an even degree meridian passing approximately through the center of the map. On this meridian lay off, as before, points which represent the distances between consecutive parallels of latitude. From Table II we may take the length of the slant height of such a tangent cone, which is tangent at the middle parallel of the map; spacing off distances on this central meridian we determine the position of the apex of the cone of development. We may then draw through the points on this meridian, which represent the various positions of the parallels, arcs of circles which will represent the developable parallels. On the middle parallel of the map we

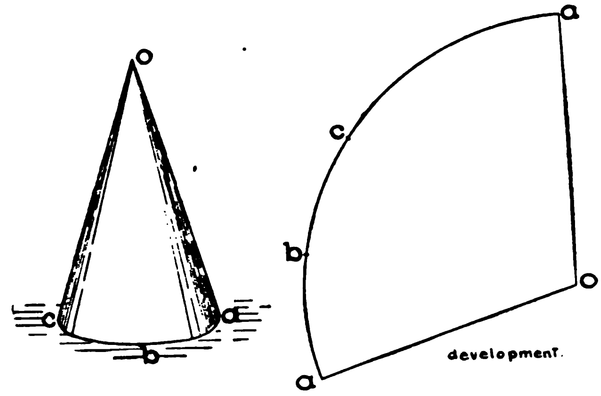


Fig. 11. Development of Cone.

may plot the position of the various meridians included in the map and through these points we may draw radial lines in the direction of the apex of the developable cone. Points in the immediate neighborhood of the middle parallel are correctly plotted. Points which are further away from the middle parallel, either above or below, are more and more inaccurate; but this method will answer for plotting of areas of several hundred miles. One very practical difficulty, which is almost insuperable unless the scale of the map is very small, is that the slant height of the developable cone is too large and the apex of the cone is really inaccessible. For example, at latitude 40° the length of the side of the tangent cone is 4730 miles. Even if we adopt a scale of 1 ÷ 125000, which is approximately two miles to the inch, the length of the radius to be used in developing this cone would be 4730 ÷ 2 = 2365 inches = 197 feet. It then becomes impracticable to use this method except by adopting a method of "co-ordinates" which will be described later.

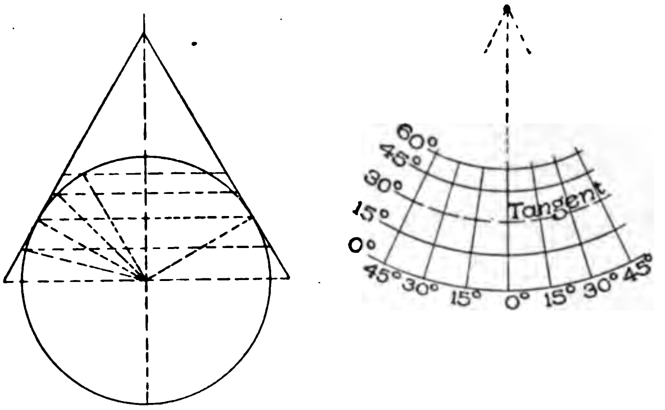


Fig. 12. Simple Conic Projections.

Simple Conic Modified. The simple conic is totally outside of the sphere, except at the circle of tangency. The map is therefore always larger than the true area. A simple modification is to lay off on two developed parallels situated at one-fourth of the height of the map from top and bottom the true distance between meridians. The meridians are then drawn as straight lines through these points of division. This is virtually the equivalent of the development of a cone which intersects the sphere between the two selected parallels, the cone being inside the sphere between the parallels and outside of the sphere above and below them. That part of the map which lies between the two parallels will be slightly too small and that above and below will be slightly too large, but in no case will the variation be as large as in the case of a simple conic. A still further modification is to lay off on each parallel the proper distance between meridians for that latitude, but the parallels are developed as arcs of concentric circles. Each of these successive methods is a little more accurate than the previous method, but unless the scale of the map is such that it is practicable to draw the developed parallels directly as arcs of circles and without the use of coordinates, it is just as simple, as well as being much more accurate, to employ the following method, which is the method used by the United States Coast and Geodetic Survey, as well as by all other geodesists.

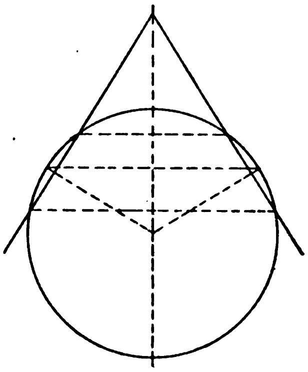


Fig. 13. Simple Conic—Modified.

Polyconic Projection. Let us consider a sphere with parallels of latitude and meridians drawn over it and then assume that the sphere is covered with a series of cones whose vertices are all in the pole of the sphere, each cone being tangent to the sphere along one of the parallels of latitude. If we then develop these cones, the line of tangency of each cone will develop as the arc of a circle whose radius is the slant height of the cone, and the intersection of the tangent parallel with each meridian may be plotted on this circle in its true position. By drawing a vertical center line we may lay off on it the true distance between the desired parallels of latitude; then, laying off on that vertical center line from the position of each parallel, a line equal to the slant height of the corresponding tangent cone, we may draw arcs which will represent the development of each parallel of latitude. To illustrate the principle involved this has been done on an exceedingly small scale and by lines which represent the development of many degrees of both latitude and longitude in Fig. 14, but this method is particularly used for the plotting of maps on a comparatively large scale in which the length of the radius of the developing cones (at the scale adopted) would be impracticably large. It therefore becomes necessary to plot these curves by the method of co-ordinates, which must necessarily be computed and arranged in a tabular form for use. It then becomes just as easy to use co-ordinates which are computed with reference to the true form of the earth rather than those computed for a sphere. In Table II is given for various latitudes the length in statute miles of 1° of latitude, the length of 1° of longitude, the length of the side of the tangent cone, the distance from the center meridian of 1°, 2°, 3°, 4° and 5° of longitude, and also the vertical ordinate for each one of these points

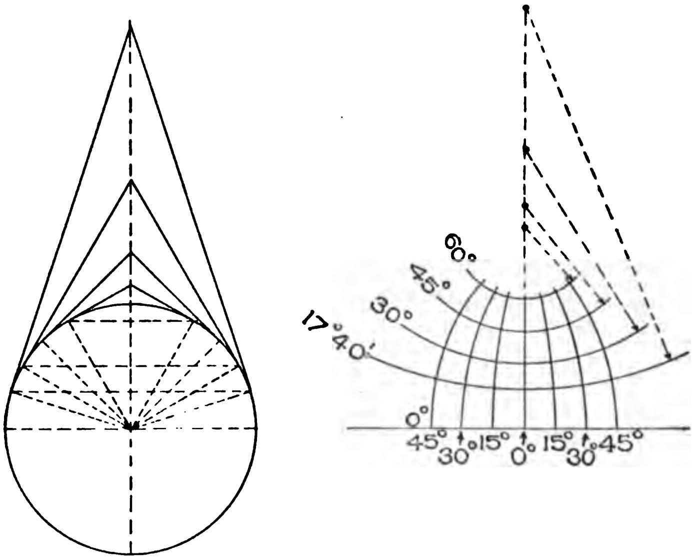
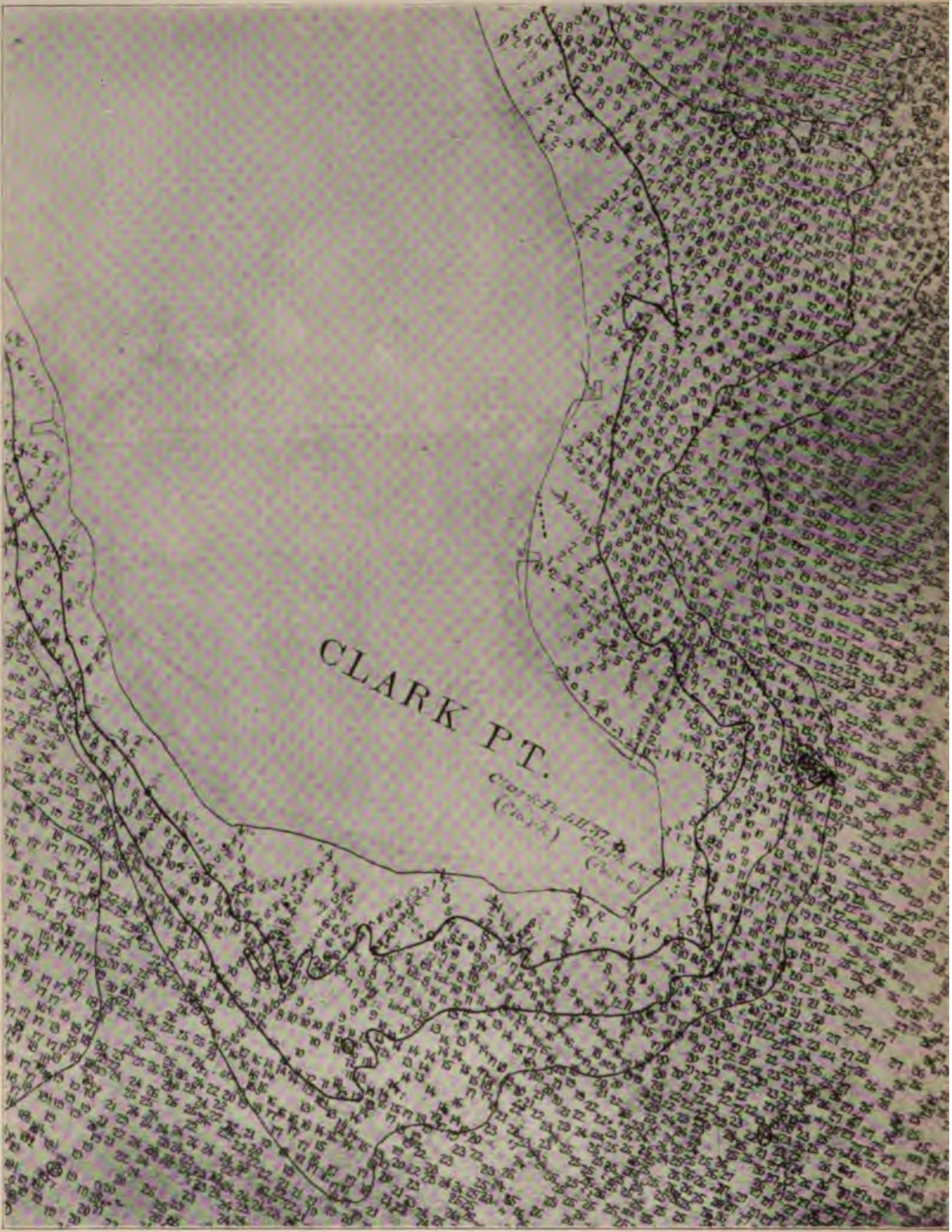


Fig. 11. Principle of Polyconic Projection.

above a horizontal tangent line. The method of drawing one of these maps may be most easily described by a concrete example, as follows: *First.* Draw a vertical line to represent some even-degree meridian of longitude which is as near as may be at the center of the map. Having selected the scale, space off on this vertical line the positions for as many parallels of latitude as it is desired to represent. The length of 1° of latitude measured on a meridian for various latitudes in statute miles is given in the accompanying table. For example: suppose that the scale of the map is to be $\frac{1}{125,000}$. This is



ILLUSTRATING ONE STEP IN CHART MAKING

Photographed from an original sheet on which hydrographic work was done in the field. Note the immense number of soundings plotted (which, however, are but a small proportion of those taken) and the fathom contour lines, and compare with finished chart shown on page 58.

approximately equal to a scale of two miles per inch. At latitude 40° the length of 1° of latitude in statute miles, as taken from the table, is 68.993. Multiplying this by 63360 we have 4371396 inches, which is the length of 1° at 40° latitude, or which is the length between 39° 30′ and 40° 30′. Dividing this by 125000 we have 34.97 inches, which will be the distance to be scaled on the vertical line between two marks, which should represent the parallels 39° 30′ and 40° 30′. Dividing this space in half we have the position for 40° latitude and any other desired parallels may be interpolated accordingly. This is, of course, on the assumption that 40° is the mean latitude. *Second.* From each point on the center line through which a parallel is to pass, draw a horizontal line. Assume that we wish to lay off meridian lines at intervals of 0° 15′ of longitude. The accompanying table, which has been compiled from the more extensive table of the United States Coast and Geodetic Survey, published in their report for 1884, gives the co-ordinates of points which are from 0° 15′ to 3° east or west from the center meridian of the map for all latitudes between 25° and 50°. These co-ordinates are given in meters since this is the universal unit of measure in high grade geodetic work, but these co-ordinates can be readily transferred into other units on the basis that one meter = 3.2809 feet = 39.3704 inches. Although the accompanying table is amply accurate for much of the work that an engineer may have to plot, the more extensive tables of the Coast Survey should be utilized for the highest grade of work. Since 1 meter = 39.3704 inches, when we are working on a scale of 1 : 125000 each meter would scale .00031496 inches or 10000 meters = 3.1496 inches. For our present purpose we may call this 3.15 inches. According to our table the distance out from the center for 0° 15′ at latitude 40° is 21349 meters, which at 3.15 inches per 10000 meters gives 6.72. Similarly we can reduce to inches the ordinates for 0° 30′, 0° 45′ and 1°. The distance out for 1° 15′ can be obtained sufficiently close by interpolation between the values for 1° and for 1° 30′. At all these points we can erect short perpendiculars. This should be done on both sides of the vertical center line. The vertical ordinates which are given in the table may be similarly changed to inches. The vertical ordinate in meters for 1° from the center is 479 meters, which at .000315 inches per meter = 0.151 inches. For 2° the vertical ordinate is four times this or 1916 meters or 0.603 inches, thus indicating that the vertical ordinates increase as the square of the distance from the meridian. Therefore the ordinates for 1° 15′, 1° 30′ and 1° 45′ will = $\frac{25}{16}$, $\frac{36}{16}$ and $\frac{49}{16}$, respectively, of the unit height for 1°, which is 0.151 inches. For a width in latitude of 3°, each side of the center, other points may be similarly obtained. Having plotted these points for 40° latitude, we must repeat the operation for each new parallel required. The figures to be used for intermediate parallels of latitude may be obtained with more than sufficient accuracy by interpolating between the numbers given in the tables.

It should be noticed that Fig. 14 was drawn mainly for the purpose of illustrating the general principles involved. It covers a range of 45° each side of the center line and from the equator to 60° north latitude. This is a very extreme case and far greater than would ever be used on any map where close accuracy is a necessity, and yet even in this case the distortion, although it does exist, and is an absolute necessity, is nevertheless reduced to a minimum by this method.

Numerical Example. Compute by derivation from Table II, for the scale of 1 ÷ 1,000,000 the values for each intersection point of parallels at intervals of 0° 15′ and meridians at every 0° 15′ for 3° east and west of the prime meridian and between the latitudes of 36° and 37°. Draw lightly in pencil the horizontal lines and the perpendiculars, indicating in pencil the computed length of the divisions of the horizontal lines and the lengths of each perpendicular, then connect the corresponding intersections with lines which will represent the parallels and meridians. Draw to a scale of 1 : 1,000,000.

22. Three Point Problem. Economy in field work is frequently served by employing what is known as the "three point problem" to accomplish certain results. For example, the location of points at which soundings are taken in any body of water require the employment of several extra men with instruments if it is done by the method which is perhaps simplest to plot, namely, to have men and instruments on certain located points on shore where observations are taken at the time of making each sounding. This may be obviated by using two sextants or even a double sextant, which is used by a man on the sounding boat. Observations by the boat observer are taken on three shore signals, and the two angles between each outer signal and the middle signal are measured. With an instrument known as a double sextant, one skillful observer can simultaneously

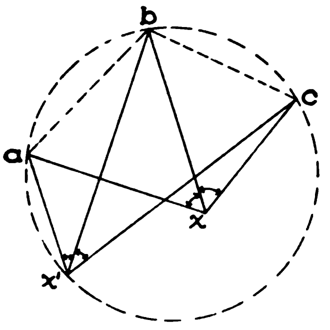


Fig. 15. Three-Points Problem.

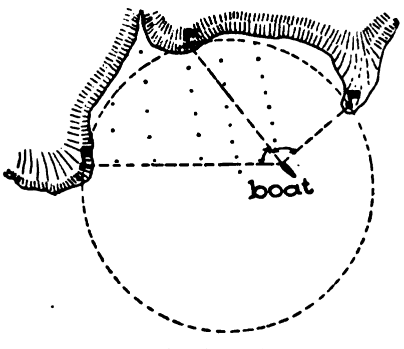


Fig. 16. Locating Soundings—Three-Points Problem.

set his sextant to take both readings. This method keeps all members of the party together. No signaling between those in the boat and instrument men on shore is necessary and the economy of time and men is considerable, although it requires special instruments to take the observations and a special instrument later on to plot the work. Unfortunately even this solution has its limitations. If the circle which passes through the three fixed points also passes through the point to be located, then the solution is indeterminable, and if the point to be determined lies *near* the circumference of that circle, the solution, although theoretically determinate, is subject to great inaccuracy. Nevertheless, with suitable care this limiting condition may be avoided and the accuracy under usual conditions is as great as is necessary. The same fundamental principle is employed in plane table surveying when it is desired to set up a plane table at some station which has not been previously plotted on the map, and to determine directly its location by an observation on three stations which have already been plotted. The fundamental principle of the work is illustrated in Figure 15 and depends on the geometrical proposition that, except for a point located on the circumference *a b c*, there is but one position, *x*, from which the lines drawn forming the angles *a x b* and *b x c* will equal any two given values. An inspection of the figure will show that if *x'* were located in the circumference, then the angles *a x' b* and *b x' c* would be the same for any location of *x'* in that circumference. If *x* represents, for example, the location of a boat from which the angles ***a x b*** and ***b x c*** are measured between flags located on shore, it is only necessary, in order to plot the position of *x*, to have three lines which make the given angles with each other and find by trial such a position for *x* that three such lines will simultaneously pass through the three points. These three lines may be the three arms of a "three-armed protractor" (see Fig. 21) or they may be three lines which make the given angles and which are drawn on a piece of tracing paper, which is used as described below. Of course, very skillful work is required to avoid inaccuracy, but in skilled hands the method is economical and sufficiently accurate for practical use.

In plane table work the problem may be solved similarly. The plane table may be set up over the station whose location is required. A piece of tracing paper may be placed over the regular drawing and three lines may be drawn to the three known and plotted stations. These lines will evidently make the proper angles with each other. It is then only necessary to move this tracing paper to some position, such that the lines will simultaneously pass through the three points and then prick through the tracing paper the location of the desired point.

Another method of solving the same problem with the plane table is that known as Bessel's solution, which is illustrated in Fig. 17. The explanation of this method is far more difficult than the mere work of doing it, which may be readily learned. The reasoning may be best followed by considering a case which has already been worked out—see Fig. 17. *a*, *b* and *c* are points already plotted on the map. *d* is a point whose real location is at first unknown. *e* is a "construction" point, which is also unknown. The method of procedure will be first stated as follows:

The instrument is set up over the point *D* in the field. *A*, *B*, *C* and *D* refer to the actual station positions. (1) The alidade is then set on the line *c a* and the whole table is then revolved until the instrument sights at *A*, when it is clamped. (2) The alidade is then placed with its edge on *c* and so that it sights at the point *B*. Drawing a line, we will have the line *c e*. (3) The alidade is set on the line *a c* and the table is then loosened and turned until the telescope points at *C*. (4) Move the alidade so that the edge passes through *a* and again points at *B* and draw a line. This line *a e* drawn from *a* intersects the line drawn from *c* in the point *e*. (5) Connect *b* and *e*. The required point *d* is somewhere on the line *b e*—or on *b e* produced. By placing the alidade on the line *b c* and turning the whole table until it points at station *B*, the table is properly oriented. The student should note that the location of point *d* is still unknown, but when the table is properly oriented, it is only necessary to place

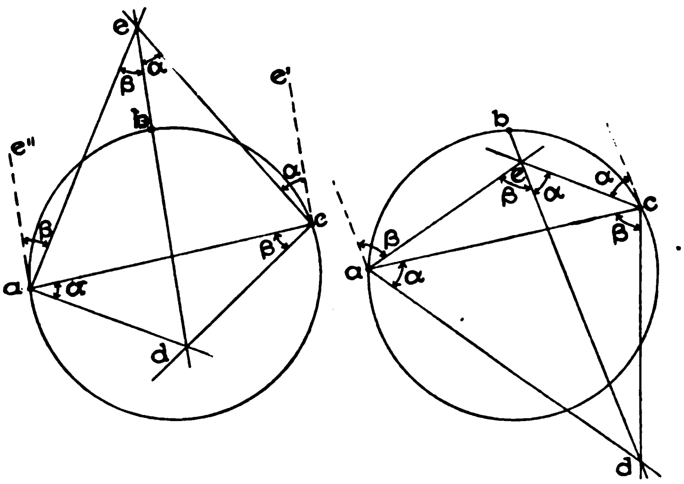


Fig. 17. Bessel's Solution of Three-Point Problem.

the alidade with its edge on the point *a* and so that it sights at station *A*. A line drawn along the alidade will intersect the line *b e* at the required point *d*. As a check we may turn the alidade so that its edge is on the point *c* and so that the telescope points at station *C*. The edge of the alidade should again pass through the point *d*; the precision with which it does so forms a valuable check on the accuracy of the work done.

Explanation. First step: alidade on *c a*, sighting at *A*. Since the instrument is at *D*, the proper direction of the alidade when sighting toward *A* should be *d a* and would be so if the table were properly oriented. Therefore the orientation is in error by the angle *a* to the *right*. Second step: the line *c e*, which should have the direction *c e'*, makes the angle *a* with *c c'*. In other words, the line *d b e* makes the angle *a* with *e c*. Third step: alidade on *a c*, sighting at *C*. Since the instrument is at *D*, the proper direction of the alidade when sighting toward *C* should be *d c*, and would be so if the table were properly oriented. Therefore the orientation is in error by the angle β to the *left*. Fourth step: the line $a\ e$ which should have the direction $a\ e''$, makes the angle β with $a\ e''$. In other words the line $d\ b\ e$ makes the angle β with $e\ a$. Fifth step: d lies somewhere on $b\ e$ or on $b\ e$ produced. Since the table can be accurately oriented by sighting on B with alidade on $d\ b\ e$, the location of d then becomes easy as a problem in "resection" by turning the alidade to sight at either A or C. The alidade will intersect $d\ b$ at d and thus determine the point. The student should note that as d approaches the circumference, e approaches b, the line $b\ e$ becomes very short and since the solution depends on drawing $b\ e$ accurately in direction, it becomes more and more inaccurate as d approaches the circumference. When d is on the circumference, e will coincide with b and the solution is indeterminate. It should also be noted that d and e are interchangeable—*i.e.*, if e were the unknown point, then d, determined by the intersection of $a\ d$ and $c\ d$, would be the point which, when joined with b, would give the line on which the true point e would be located. e would then be determined by resection from a or c.

This method has the theoretical inaccuracy of assuming that lines drawn from various points of the map toward a distant station are parallel. When the scale is very large the error *might* be appreciable with the most careful work, but ordinarily it is impossible to do plane table work with sufficient exactness to detect an error due to this cause. The advantage of this method over the tracing paper method is that although it requires the drawing on the paper of the lines $a\ e$, $b\ e$, and $c\ e$, which are otherwise useless, it does not necessitate the use of tracing paper which might prove inaccurate on account of slipping. When it is found that the unknown point d is nearly, if not exactly, on the circumference of a circle passing through a, b and c, then some other station should be substituted for a, b or c, and ordinarily this will permit the problem to be solved with sufficient accuracy.

23. Topographical Signs. There is a very great advantage in employing colors to represent the topographical signs, but if a map is being prepared for reproduction it is impracticable to use colors unless it is designed to make several impressions, one in each of the various colors, which makes a very expensive map. The systems of topographical signs must therefore be based very largely on whether colors are to be used or whether the whole map must be in black. When one individual map is being made, it is almost as easy to employ colored inks, and this should certainly be done if the map is designed to represent considerable variety of detail. Contour lines are usually drawn with "burnt sienna," a yellowish-brown color which is not so very unlike that of clay and which is quite suggestive. In spite of the added expense, the United States Geological Survey maps have all contours printed in a similar color. Similarly water lines are usually represented with blue lines if color is employed. The United States Geological Survey maps represent the shores of all lakes, rivers, etc., with blue lines. Even the smallest streams which are represented on the map are drawn in blue even though it is but a single line, and swampy districts are represented by a suitable topographical sign, which is also drawn in blue. This practice permits an instant recognition of the character of the sign and prevents any possible confusion of mistaking a stream line for a contour line. Artificial features of the topography are represented in black. By artificial features are meant anything in the nature of constructions such as buildings, railroads, dams, property lines or political boundaries, etc. As stated previously under "scale of map," judgment should be shown whether to endeavor to represent the actual form of buildings to their proper scale. On the United States Geological Survey maps individual dwelling houses are usually represented by a minute square of black even when the scale of the map is as small as one mile to the inch (approximately). Since the maps are drawn very finely and the squares are perhaps not more than $\frac{1}{100}$ of an inch square, they do not necessarily indicate much, if any, exaggeration. Of course, at such scale no attempt is made to give the exact form of the building. The well known representation of a railroad by a single heavy line crossed at frequent intervals with cross lines as if in imitation of cross ties is always recognizable. When the scale is sufficiently large, say 100 feet to the inch, even the two rails may be indicated by a double line rather than a single line. Highways are indicated by double lines which should be spaced at approximately their proper width. Since a neat appearance absolutely requires that these lines should be evenly spaced and exactly parallel, it is preferable to draw these lines with what is called a "road pen," which consists of two ruling pens whose distance apart can be regulated with a screw. Using such a pen even a rough and irregular road may be rapidly drawn in so that both lines will not only be uniform, but are exactly the same distance apart at every point.

Some instruction books have been published with very elaborate systems of topographical signs to represent by distinctive signs every conceivable fcrm of vegetation and also to denote the character of different kinds of factories, mines and other structures. In several cases where a map is made for the distinctive purpose of denoting the location of the various features which pertain to some special industry, these special topographical signs may be justifiable, but their significance should invariably be indicated by a suitable legend inscribed on some part of the map. Although every surveyor should know the significance of the topographical signs which are used to indicate the common features which are generally found on every map, it is useless to expect that the special and little used topographical signs will be familiar to all engineers. Plates I and II show the signs which have been adopted by government use and which therefore may be considered as standard. No others should be placed on a map without some lettering on the map to indicate their character, especially as there is some confusion regarding the method of indicating some of the more uncommon topographical features.

24. Lettering. Many a map which is otherwise good is spoiled by poor lettering. Part III of Mechanical Drawing contains a treatise on lettering, and therefore such instruction will not be repeated here, since this course forms a part of every engineering course. In general it should be said that the style of lettering adopted should be simple and as plain as possible. No fancy lettering should be used except perhaps in the title, and even here some forms of fancy lettering are not only in bad taste but are a useless waste of time and skill. The illustrations of government maps herewith given should be particularly studied with reference to the style of lettering used for different topographical features. Where much lettering is necessary (for example, to indicate the location of numerous towns on a small scale map) the simplest form of line lettering is preferable. When possible all lettering should run horizontal with the base of the map, except the lettering to indicate the name of streams, railroads, etc., which should follow their course. The spacing between letters used to indicate the names of streams and railroads should likewise be strung out to cover a considerable part of the length of the stream or railroad unless that spacing would be so great as to make it difficult to follow the continuity of the letters. The Roman style of lettering is usually adopted on government work, because it is the best in spite of its being the most difficult style to make so that it looks well. The student should cultivate the ability to make lettering in this style. A simple form of line lettering is infinitely preferable to an attempt at fancy lettering, and the draftsman must not consider that he is qualified to do lettering until he can make good letters at least in this simple form.

25. **Border Line.** A simple border line which consists of a single heavy line of uniform width will add considerably to the neatness of a map, and it should usually be made. When it is specially desired to attract the eye of non-technical people, as in the case of real estate maps, which are made to look as attractive as possible to catch the eye of a possible purchaser, a very elaborate border line may be justifiable, but otherwise it must be considered as a waste of time and skill, and even an evidence of bad taste. It sometimes happens that a farmer will think a great deal more of the map of his farm if it is surrounded by an elaborately executed border line, and since it is policy to sell to a man what he wants and is willing to pay for, it would be justifiable in such cases to consult a book on lettering, which usually contains various specimens of border lines, meridian marks, etc., which will set off a map and make it look very pretty even though it does not add one whit to its real value. The sheets issued by the United States Geological Survey have no border line, and even the large sheets issued by the United States Coast and Geodetic Survey have border lines which are simple and dignified in character.

26. **Drawing Paper.** Considering that the work on very large maps is very great and sometimes requires a very high grade of labor for weeks and even months, which means that the map represents an expenditure of labor worth several hundred dollars, it is folly to lessen the value of the map by using a cheap grade of paper. A good quality of drawing paper should be quite smooth and yet of such a texture that pencil lines and even light ink lines can be erased from it without necessarily destroying the surface of the paper. For some kinds of work where but very little penciling is required and such penciling as is put in is almost entirely covered up with ink lines, it

may be desirable to use Bristol board, since its surface is very smooth and, with very fine pens, work may be done which is almost equal to engraving, but Bristol board becomes roughened when it is rubbed, and therefore it should not be used when many construction lines are necessary which need to be more or less erased. "Hot pressed" Whatman's paper is the best for map work, since it is smoothest and it can be obtained in large sheets. When it is necessary to make very large maps (four feet square or over) it is advisable to have the paper "mounted" on linen. This becomes almost necessary on account of its greater durability, for a large sheet would be more apt to be torn. Manufacturers of drawing paper make these large sheets by pasting smaller sheets together on the mounting of linen, the joints being beveled and carefully rubbed down so that they are practically invisible and do not give the roughness that would ordinarily be found with an ordinary lap joint. Since such work is done so much better by the manufacturers, it is usually preferable to leave such work in their hands. When it is absolutely essential for one to do his own paper mounting, it may be done as follows: Stretch the cloth on a drawing board, if a sufficiently large one can be found, or even on a smooth floor, and tack it down on all sides, stretching it as much as possible, so that it is perfectly smooth. Trim the edges of the sheets of paper (say four) which are to be joined together, and with a rubber grind down the edges which are to form the joint until they get literally to a feather edge. Cover each sheet thoroughly with paste and then spread it on the cloth as smoothly as is possible in its puckered condition, taking special care that the edges of the sheets overlap properly and that a perfectly smooth joint is made. Sometimes a warm flat iron may be successfully used while the paper is drying to smooth out any wrinkles that have formed. By watching the paper during the process of drying and carefully smoothing out wrinkles until all have disappeared, and then leaving the paper for at least twenty-four hours until it is perfectly dry, it will usually be found that the sheet will be perfectly smooth when it has dried. Only the best quality of paper should be used for such a purpose.

DRAWING INSTRUMENTS.

The chief drawing instruments required for topographical work are very few and simple. A protractor, a scale and a straight edge, with pens and pencils, almost comprise the list—except a few instruments which are used for special purposes.

27. Protractor. The choice of a protractor has already been discussed and it was pointed out that the very small metal protractor frequently included with a set of drawing instruments is almost useless for any practical purpose. It is so small that it is impossible to use it with any degree of accuracy. Celluloid or "xylonite" protractors with a diameter of 6 inches are very useful for plotting topography when the scale of the map is such that the length of the lines plotted is not more than 5 or 6 inches, or say twice the radius. Paper protractors, 14 inches in diameter, printed on Bristol board, which can be bought for 40 cents, are much more useful, and with reasonable care will last long enough to fully pay for their cost. Colby's protractor has the advantage that it can be permanently fixed on the paper by means of weights resting on the ears that project from the circle and a very large number of points may be plotted from one station without the wearisome endeavor to hold an ordinary protractor continuously in a fixed place. Additional scales can be attached to the central rotating plane so that any point can be immediately plotted at its proper distance from the center unless it happens to lie immediately under the circular part of the protractor.

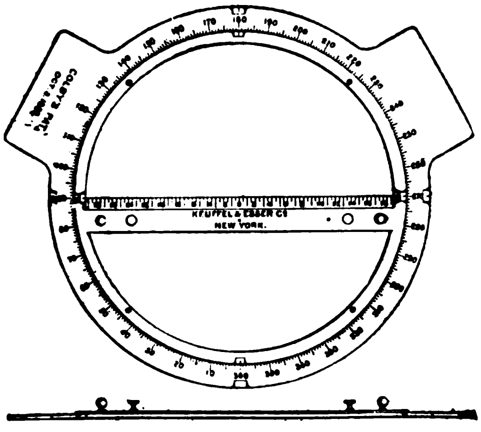


Fig. 18. Colby's Protractor.

The instruments cost about $60 with $3.50 extra for extra scales. Metal protractors of the Crozet type have the very great advantage that they can be used in connection with a T-square and do not need to be centered over the station point. After setting the protractor at the proper angle, it is only necessary to slip it along the T-square or along the fixed straight edge, which has been properly clamped on the board, and when any point of the fiducial edge of the protractor reaches the given station, a line may be drawn through the station at the desired angle. Some of these protractors are made with vernier and slow motion screws, so that they will read single minutes of arc, but it requires a degree of accuracy in the drafting work, which is seldom, if ever, attained to draw the lines as accurately as this implies. Such a protractor costs about $40. The Darling, Brown and Sharpe protractor works on the same fundamental principles as the Crozet protractor; it has a vernier reading to 5 minutes of arc and costs only $6.50. It is practically as useful and as accurate as the more expensive forms.

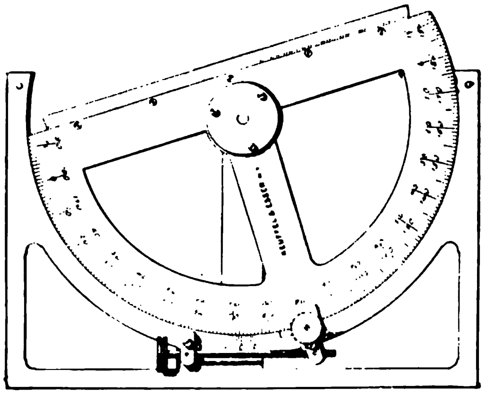


Fig 19. The Crozet Protractor.

28. Scales. Usually "decimal" scales are used for plotting topographical work, since the maps are frequently drawn on a scale of 100 feet, 200 feet, 400 feet, or 500 feet per inch. Therefore a "triangular" decimal scale with its six scales graduated to 10, 20, 30, 40, 50 and 60 parts per inch is in general the most useful for that purpose.

Even if dimensions are scaled off in hundredths of an inch, the simplest plan is to use the "50" scale in which each division means $\frac{1}{50}$ = .02 inch. When maps are drawn to some special scale, as for example, 1: 125000, or 1: 62500, it will facilitate the work (and it will be justifiable) to construct special scales which will permit the distance in feet as measured in the field to be scaled off directly on the map. There is a theoretical argument in favor of using paper scales, the argument being that the paper of the map and the paper scales will vary equally with changes in the hygrometric conditions of the atmosphere. But it must be considered that these changes are very small and almost insignificant unless dampness of the atmosphere is so great that the paper of the map becomes positively puckered, and under such conditions it is perhaps inadvisable to attempt to do any work. Also unless the paper scale was made of the same quality of paper as the map, which is not very likely, the coefficient would not be the same. It is thus seen that the advantages of using paper scales are more fancied than real, while their disadvantages are many. Where the work to be done is very extensive and it must all be done at the same scale, there is a considerable advantage in using a "flat" scale rather than a "triangular" scale, since under such conditions the five other

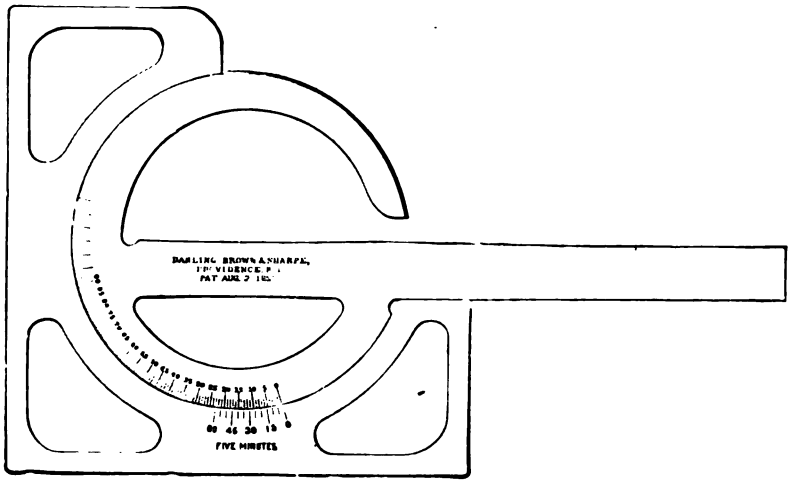


Fig. 20. Steel Protractor.

scales of the triangular scale are useless and a blunder is frequently caused by inadvertently using the wrong scale. Metal scales have the advantage of extreme durability but the disadvantage of tending to soil the paper. The boxwood scales having a natural wood color are the most common, but they grow darker with age and with handling and the graduations become so dim that it is difficult to use them. Ivory scales are very liable to warp and shrink, and although they have the advantage of distinctness and a neat appearance, their tendency to warp is a decided disadvantage. Probably the best scales are those which are made of boxwood but which have the beveled edges covered with a material which resembles ivory, which permanently remains white, and which does not shrink as ivory does.

29. **Straight-Edge.** It is not a good plan to use the edge of the scale as a straight-edge. The rubbing of the pencil over the edge of the scale will soil it. It is always better to have two triangles, so that lines at right angles may be readily drawn in as is necessary in showing the sides of buildings. A very long straight-edge is occasionally necessary. When a very long line is to be drawn perfectly straight and no straight-edge of that length is at hand, the line may be drawn by stretching a silk thread between the two terminal points and then marking, with a pencil, points at such frequent intervals that the small straight-edge available may span those distances. It is needless to say that extreme care is absolutely essential in drawing a line by this method. Even well-made straight-edges may become warped and lose their straightness. To test a straight-edge, carefully draw a line with it and reverse the straight-edge end to end and note how closely it coincides with the line drawn. If no error is observable, the edge is probably straight, but it is conceivable that a straight-edge might not be straight and yet stand this test, for the edge might have a form somewhat similar to an *S*, that is, it might be curved symmetrically about its center so that when reversed the edge would again coincide with the line drawn, but this is a very improbable condition.

30. **Pens and Pencils.** The lead used in the pencils should be of good quality and should be neither too hard nor too soft. A very soft pencil will have its point worn off very rapidly and the lead will smudge the paper. If the lead is too hard, it requires so much pressure on the paper to produce a visible mark that the paper is

actually indented, and if it becomes necessary to erase the line, the paper is indelibly impressed with a groove where the line was drawn. It is therefore necessary that all lines and marks should be drawn with a very light pressure of the pencil, which practically requires that the lead should be soft enough so that even a light pressure will make a mark that is easily visible. Although some of the inking work is done with a ruling pen, the great bulk of topographical mapping is done with a nib pen. The pen should be sharply pointed, so that it is capable of making a very fine line. Although much of such work is done with "crow quill" pens, a larger pen (say Gillott's No. 303) will serve equally well and in some respects is more useful. The rubber used should be a soft rubber rather than hard, since hard rubber will scratch the paper. A "sponge rubber" to clean the drawing when it is finished is almost a necessity.

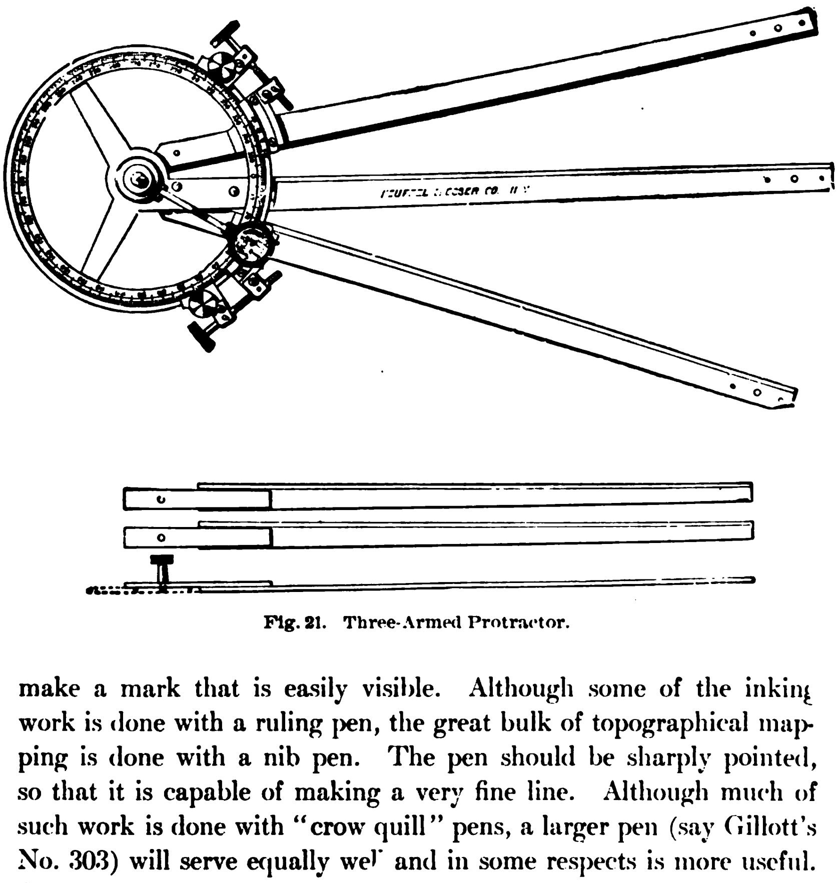


Fig. 21. Three-Armed Protractor.

31. Three-Armed Protractor. Among the special drawing instruments which are occasionally needed is a "three-armed protractor." This instrument is used when plotting points which have been observed on the principle of the "three-point problem." The instrument consists essentially of a protractor having one fixed arm and two movable arms which may be set at the two given angles with the middle arm. Then when the fiducial edges of these three arms are simultaneously placed on the three given points, the center of the instrument *must* be located at the point which represents on the map the point from which observations were taken, and this point is pricked through the center of the instrument. As previously explained, the operation of this instrument depends on the fundamental principle that (unless the point to be located happens to lie in the circumference of the circle which passes through the three given points sighted at) there is only one position from which it is possible to draw three lines which make the given angles with each other through those three points. The angles are therefore laid off by moving the two outer arms so that they make the required angle with the middle fixed arm; then with the arms clamped in position, the whole instrument is shifted around on the paper until the fiducial edges of the rules simultaneously pass through the three fixed points. It is needless to say that great care and accuracy is necessary, and especially so if the point to be determined lies near the circumference of the circle passing through the three fixed points, for it will be found that a very slight inaccuracy

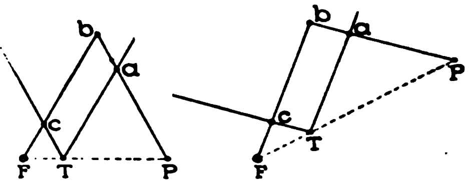


Fig. 22. Principle of Pantograph.

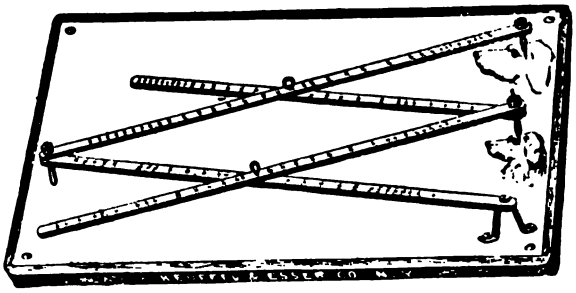
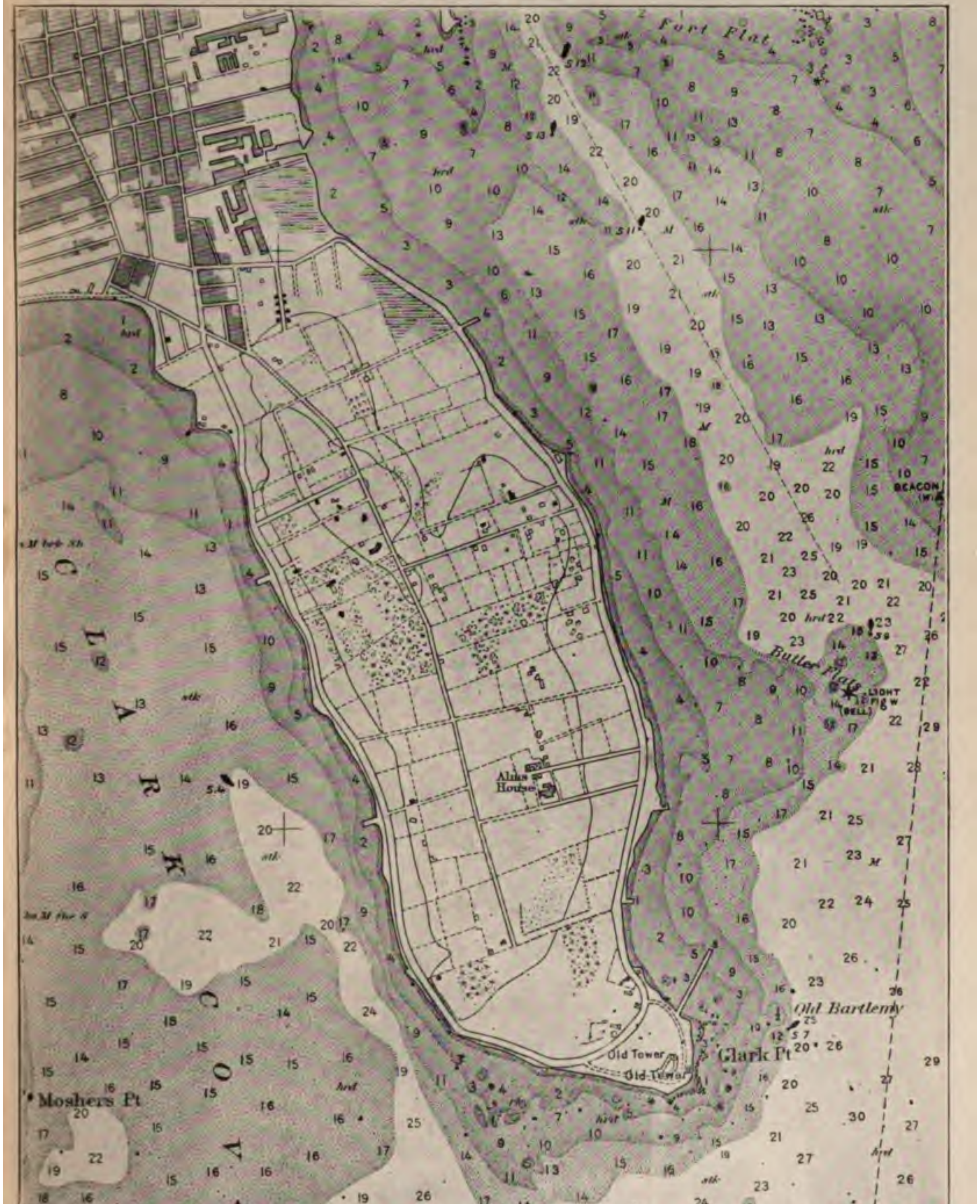


Fig. 23. Wooden Pantograph.



SECTION OF FINISHED HYDROGRAPHIC CHART

Showing leading topographical features of land surface, both above water and submerged. Note soundings and fathom contour lines. For preliminary stages in preparation of this chart, see pages 26 and 42.

will cause a considerable variation in the location of the unknown point; or in other words, there will be a considerable area from any point of which the three lines will pass approximately through the three fixed points.

32. Pantograph. The pantograph is a very useful and essential instrument when drawings are to be re-drawn on a different scale. It frequently happens that field drawings, especially when made with plane table, are drawn to a larger scale than is used for the final engraved maps. The principle of the pantograph is as shown in Fig. 22, where *F* represents the fixed point, *T* a tracing point and *P* a pencil or marker. *F*, *T* and *P* must *always* be in a straight line. *F c T* and *F b P* must always be *similar* triangles. In this instrument they are *isosceles* triangles. *a b c T* is always a parallelogram. The ratio *P F* : *T F* is the ratio of enlargement. To reduce, transpose *P* and *T*; *i.e.*, the marker will be placed at *T* and the tracer at *P*. The diagram shows the fundamental principles, but in practice accuracy requires very fine workmanship and an expensive instrument. Very simple instruments are sometimes made by combining four sticks as shown in the sketch, using rivets at the joints, but the friction of the joints makes such simple instruments inaccurate. Fig. 23 shows an instrument in this simple form, which may be bought for $1.75. Fig 24 shows another instrument called a precision pantograph, which has the same essential geometrical principle, although the mechanical construction has a somewhat different outline and the material, which is entirely of metal, has the very finest quality of workmanship. The cost varies with the length of the bars, from $155 to $165.

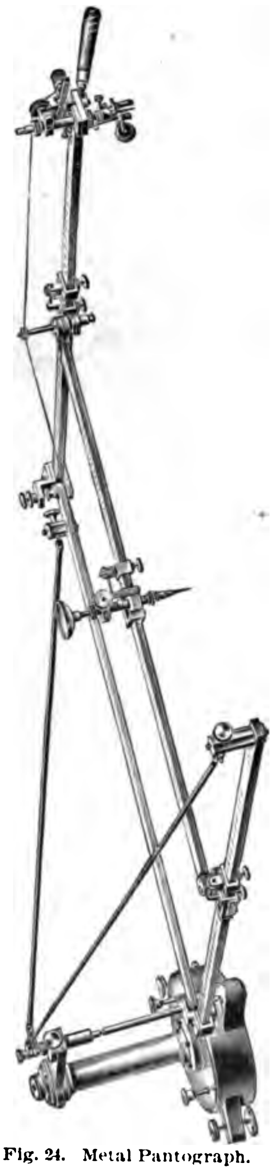


Fig. 24. Metal Pantograph.

33. Stadia Slide Rules. Figs. 25, 26, 27 show three forms of stadia slide rules, the use of which is described in Sections 15 and 16. Any surveyor who has any great amount of stadia surveying to do will be wise to utilize the stadia slide rule to facilitate the stadia reduction, since the cost of the instrument is saved in a very few days and the money necessarily wasted by a neglect to employ this device is very false economy.

34. Polar Planimeter. This instrument is used to measure directly from any drawing the area of a given figure no matter how irregular it may be. Although they are somewhat expensive, the one illustrated in Fig. 28 costing $28, they are capable of marvellous exactness, and, considering the simplicity of their mechanism, it is a wonderful mathematical invention that such a simple mechanism would permit the recording on an index of the exact area of an irregular figure simply by tracing the pointer around the perimeter of the figure no matter how irregular it may be. The polar planimeter, as its name implies, is worked around some fixed point, called the "pole", which in practice means a needle point having a shoulder which is pricked into the paper exactly as is done with the needle point of a compass. The planimeter consists of two arms, the *pole arm*, which carries the fixed point and which is hinged to the *tracer arm*, which carries the tracing point and the measuring wheel with

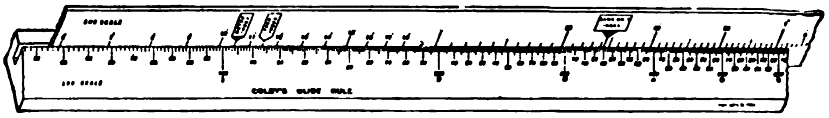


Fig. 25. Colby's Stadia Slide Rule.

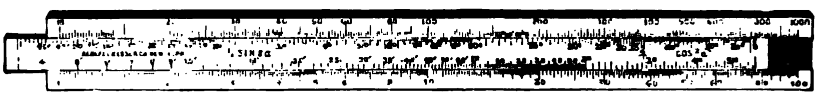


Fig. 26. K. & E. Stadia Slide Rule.

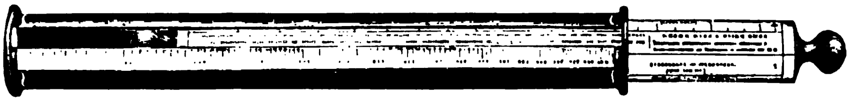


Fig. 27. Webb's Stadia Slide Rule.

the mechanism for indicating the area. The hinge will permit the tracing point to be moved away from the pole by an amount nearly equal to the combined length of the two arms, and if it is possible to fix the pole at some point from which every point in the perimeter of the figure may be reached by the tracer, then the area of the figure may be computed by a single setting of the instrument. If the figure is too large for this, then the area may be divided by lines, which are preferably straight, although not necessarily so, into two or more partial areas, and the area of each section may be separately computed. Perhaps the most common use of the polar planimeter is to compute the area of indicator cards. This has already been described in the course of instruction in steam engineering under the title "Steam Engine Indicators", but the instrument also has another important use in the measurement of irregular ground areas, especially those which have been plotted by stadia methods rather than by the courses and distances of the boundary lines and which therefore cannot be computed by the common rule of a summation of products of latitudes and departures. It is thus possible to compute an area by surveying it rapidly by the stadia method, plotting it with desired accuracy at a suitable scale and then measuring the area with a polar planimeter.

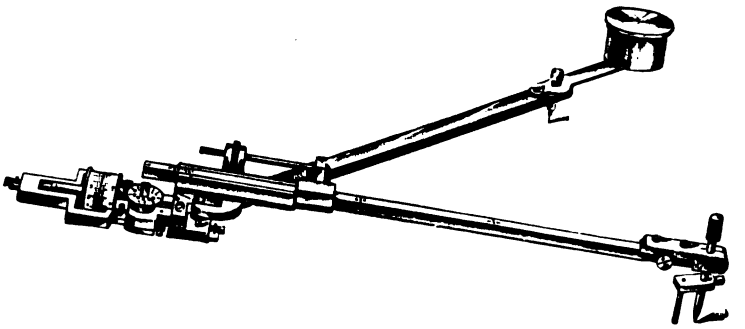


Fig. 28. Polar Planimeter.

THEORY OF THE POLAR PLANIMETER.

1. **Zero Circle.** When the planimeter is in the position *P H W C*, Fig. 29, the plane of the wheel, which is perpendicular to the axis *P H*, passes through *C*. If the instrument is revolved about *C*, with the angle *W H C* ($= a_0$) always *constant*, the motion of the wheel over the paper will have no component in the direction of its

plane and the wheel will not revolve. The pointer in this position describes the "zero circle".

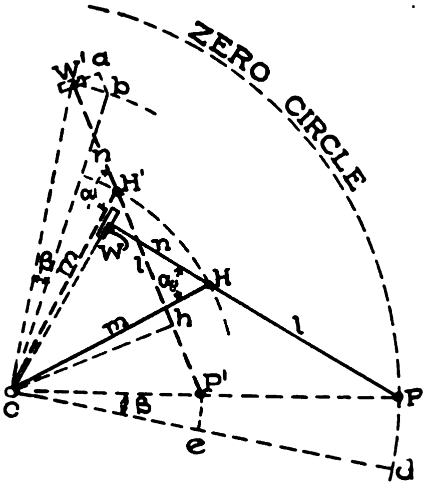


Fig. 29. Polar Planimeter Diagram.

2. Combined Sliding and Rolling. When the planimeter is in the position P' H' W' C and is revolved about C, with the angle W' H' C $(= a_1)$ always *constant*, the wheel will have a combined sliding and rolling motion. For an infinitesimal movement W' b the wheel will roll an amount W' a and slide perpendicular to its plane an amount a b. When rolling in this direction, the movement is called *negative*.

3. Radial Motion. When the point P is moved from P to P', the wheel W will both slide and roll, but its rolling will all be in a *negative* direction.

4. Reversed Radial Motion. If the pointer were to move back from P' to P the wheel would again slide and roll in precisely the same amounts but in contrary directions, and when it reached P it would have identically the same position and the reading of the index would be identical with the previous reading at P.

5. If the pointer were to move from e to d, the amount and direction of both the slipping and rolling would be the same as when it moved from P' to P.

6. If the pointer were to start from P, move to P', thence to e, thence to d, and thence back to P, the resultant rolling of the wheel is the same as that for the line P' e alone; for the rolling for d P is zero (¶ 1) and the rolling for P P' will be just neutralized by that for e d. (¶'s 3, 4 and 5.)

7. Therefore when the pointer is moved to the right on the arc of a circle within the zero circle about C as a center the indication is *negative* and is the same as if the pointer moved around the area

included between the arc, the corresponding arc of the zero circle, and the including radii.

8. If the pointer moved in the opposite direction the indication would be the same in amount but *positive*.

9. By similar demonstrations, similar facts may be shown for any other elementary area except that

(*a*) when the pointer is *outside* the zero circle and moving to the *right*, the indication is *positive*, and

(*b*) when *outside* and moving to the *left*, the indication is *negative*.

10. The perimeter of any area may be considered as made up of a combination of *infinitesimal* arcs and radial lines having the fixed point of the planimeter as center. Its total area is the *algebraic* sum of all the infinitesimal areas lying between *each* arc and the zero circle.

11. If the pointer of the planimeter moves around each infinitesimal area in turn in such manner that when moving on the perimeter it moves in the same direction as though moving continuously around the perimeter only, the pointer will move over all interior lines an *even* number of times in *opposite* directions. Therefore the *accumulated* registration of the wheel will be the same as though it moved on the perimeter only, for all registrations on interior lines will be nuetralized by the equal motion on them in opposite directions. (¶'s 4 and 6.)

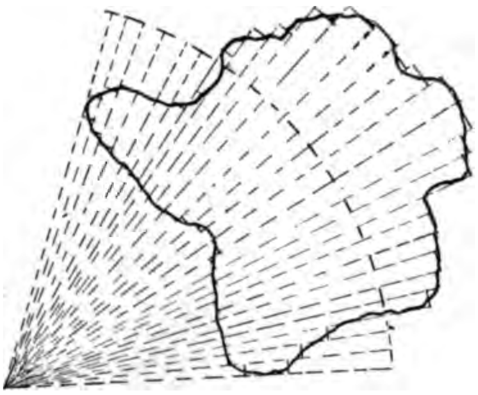


Fig. 30. Irregular Figure Divided Into Elementary Radial Areas.

12. Referring to Fig. 29, $P'c = CP' \times \beta =$

$$\sqrt{m^2 + l^2 + 2\,ml \cos a_1} \times \beta.$$

$W''b = CW' \times \beta$. The rolling of the wheel $= W''a$ (¶ 2).

$$W''a = \frac{W''h \times W''b}{CW'} = (n - m \cos a_1)\,\beta, \text{ since } \frac{W''b}{CW'} = \beta$$

and $W''h = (n - m \cos a_1)$

$$\text{Area } P\,P'\,c\,d = \frac{1}{2}(\overline{PC} \times \beta)\ \overline{P'C} - \frac{1}{2}(\overline{P'C} \times \beta)\ P'C$$

$$= \frac{1}{2}\ \beta\,(PC^2 - P'C^2)$$

$$= \frac{1}{2}\,\beta\left((l^2 + n^2 + 2nl) + (m^2 - n^2) - (m^2 + l^2 + 2ml\cos a_1)\right)$$

$$= \beta l\,(n - m\cos a_1)$$

$= l \times W'a$ (i.e., l times the rolling of the wheel.)

13. When the pointer moves around an elementary area bounded by an arc, by the corresponding arc of the zero circle and by the two bounding radial lines (all having "C" as center) the resultant motion of the wheel is the same as though it moved on the arc alone (¶ 6); the wheel rolls a distance equal to the area divided by "l" (¶ 12). If it moved in turn around each elementary area of a large area, the resultant motion of the wheel would be the same as though it moved continuously around the perimeter of the larger area (¶'s 10, 11) and therefore the total resultant motion of the wheel will equal the area of the figure divided by "l".

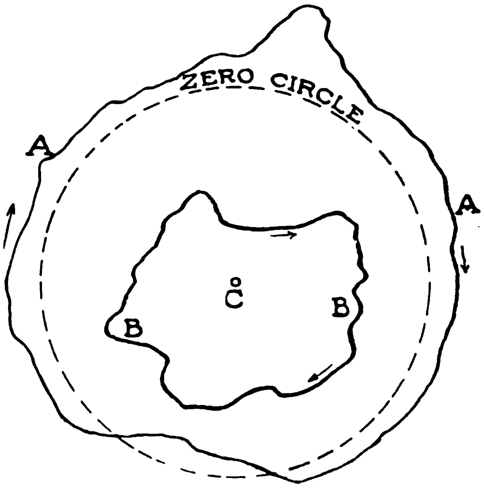


Fig. 31. Fixed Center Inside Area.

14. Therefore if c = the circumference of the wheel, n the number of turns recorded by the index, and l the length of the arm from H to P, then Area $= l\,n\,c$.

15. Fixed Center Inside the Figure. If the pointer is moved around the perimeter A, Fig. 31, to the *right*, the indication will be *positive* (¶ 9) but will indicate only the area between A and the zero circle. Therefore the total area will equal the indicated area ($l\,n\,c$) plus the area of the zero circle $\left(\pi\,(\,m^2 + l^2 + 2\,n\,l\,)\right)$. If the pointer is moved to the *right* around perimeter B, the record will be *negative* (¶ 9) and will correspond to the area between B and the zero circle. Therefore the *algebraic* sum (the numerical difference) of the record reading and the area of the zero circle will give the true area of B.

16. General Rule. *Always* move the pointer to the *right.* If "C", the pole, is within the figure, add the area of the zero circle (algebraically) to the indicated result. If the pole is outside the figure the area is given by the product $l\ n\ c$. The scale on the recording wheel is so graduated as to give the result directly. The area of the zero circle is usually indicated on the instrument, but it may be determined experimentally by finding the distance from the pointer to the fixed center, which will cause no rotation of the index wheel when the pointer is swung about the center. This is the radius of the zero circle. Its area should be reduced to the scale of the drawing.

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	8°		9°		10°		11°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	98.06	13.78	97.55	15.45	96.98	17.10	96.36	18.73
2	98.05	13.84	97.53	15.51	96.96	17.16	96.34	18.78
4	98.03	13.89	97.52	15.56	96.94	17.21	96.32	18.84
6	98.01	13.95	97.50	15.62	96.92	17.26	96.29	18.89
8	98.00	14.01	97.48	15.67	96.90	17.32	96.27	18.95
10	97.98	14.06	97.46	15.73	96.88	17.37	96.25	19.00
12	97.97	14.12	97.44	15.78	96.86	17.43	96.23	19.05
14	97.95	14.17	97.43	15.84	96.84	17.48	96.21	19.11
16	97.93	14.23	97.41	15.89	96.82	17.54	96.18	19.16
18	97.92	14.28	97.39	15.95	96.80	17.59	96.16	19.21
20	97.90	14.34	97.37	16.00	96.78	17.65	96.14	19.27
22	97.88	14.40	97.35	16.06	96.76	17.70	96.12	19.32
24	97.87	14.45	97.33	16.11	96.74	17.76	96.09	19.38
26	97.85	14.51	97.31	16.17	96.72	17.81	96.07	19.43
28	97.83	14.56	97.29	16.22	96.70	17.86	96.05	19.48
30	97.82	14.62	97.28	16.28	96.68	17.92	96.03	19.54
32	97.80	14.67	97.26	16.33	96.66	17.97	96.00	19.59
34	97.78	14.73	97.24	16.39	96.64	18.03	95.98	19.64
36	97.76	14.79	97.22	16.44	96.62	18.08	95.96	19.70
38	97.75	14.84	97.20	16.50	96.60	18.14	95.93	19.75
40	97.73	14.90	97.18	16.55	96.57	18.19	95.91	19.80
42	97.71	14.95	97.16	16.61	96.55	18.24	95.89	19.86
44	97.69	15.01	97.14	16.66	96.53	18.30	95.86	19.91
46	97.68	15.06	97.12	16.72	96.51	18.35	95.84	19.96
48	97.66	15.12	97.10	16.77	96.49	18.41	95.82	20.02
50	97.64	15.17	97.08	16.83	96.47	18.46	95.79	20.07
52	97.62	15.23	97.06	16.88	96.45	18.51	95.77	20.12
54	97.61	15.28	97.04	16.94	96.42	18.57	95.75	20.18
56	97.59	15.34	97.02	16.99	96.40	18.62	95.72	20.23
58	97.57	15.40	97.00	17.05	96.38	18.68	95.70	20.28
60	97.55	15.45	96.98	17.10	96.36	18.73	95.68	20.34
(f+c)=0.75	0.74	0.11	0.74	0.12	0.74	0.14	0.73	0.15
(i+c)=1.00	0.99	0.15	0.99	0.16	0.98	0.18	0.98	0.20
(f+c)=1.25	1.23	0.18	1.23	1.21	1.23	0.23	1.22	0.25

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	12°		13°		14°		15°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	95.68	20.34	94.94	21.92	94.15	23.47	93.30	25.60
2	95.65	20.39	94.91	21.97	94.12	23.52	93.27	25.05
4	95.63	20.44	94.89	22.02	94.09	23.58	93.24	25.10
6	95.61	20.50	94.86	22.08	94.07	23.63	93.21	25.15
8	95.58	20.55	94.84	22.13	94.04	23.68	93.18	25.20
10	95.56	20.60	94.81	22.18	94.01	23.73	93.16	25.25
12	95.53	20.66	94.79	22.23	93.98	23.78	93.13	25.30
14	95.51	20.71	94.76	22.28	93.95	23.83	93.10	25.35
16	95.49	20.76	94.73	22.34	93.93	23.88	93.07	25.40
18	95.46	20.81	94.71	22.39	93.90	23.93	93.04	25.45
20	95.14	20.87	94.68	22.44	93.87	23.99	93.01	25.50
22	95.41	20.92	94.66	22.49	93.84	24.04	92.98	25.55
24	95.39	20.97	94.63	22.54	93.81	24.09	92.95	25.60
26	95.36	21.03	94.60	22.60	93.79	24.14	92.92	25.65
28	95.34	21.08	94.58	22.65	93.76	24.19	92.89	25.70
30	95.32	21.13	94.55	22.70	93.73	24.24	92.86	25.75
32	95.29	21.18	94.52	22.75	93.70	24.29	92.83	25.80
34	95.27	21.24	94.50	22.80	93.67	24.34	92.80	25.85
36	95.24	21.29	94.47	22.85	93.65	24.39	92.77	25.90
38	95.22	21.34	94.44	22.91	93.62	24.44	92.74	25.95
40	95.19	21.39	94.42	22.96	93.59	24.49	92.71	26.00
42	95.17	21.45	94.39	23.01	93.56	24.55	92.68	26.05
44	95.14	21.50	94.36	23.06	93.53	24.60	92.65	26.10
46	95.12	21.55	94.34	23.11	93.50	24.65	92.62	26.15
48	95.09	21.60	94.31	23.16	93.47	24.70	92.59	26.20
50	95.07	21.66	94.28	23.22	93.45	24.75	92.56	26.25
52	95.04	21.71	94.26	23.27	93.42	24.80	92.53	26.30
54	95.02	21.76	94.23	23.32	93.39	24.85	92.49	26.35
56	94.99	21.81	94.20	23.37	93.36	24.90	92.46	26.40
58	94.97	21.87	94.17	23.42	93.33	24.95	92.43	26.45
60	94.94	21.92	94.15	23.47	93.30	25.00	92.40	26.50
(f+c)=0.75	0.73	0.16	0.73	0.17	0.73	0.19	0.72	0.20
(f+c)=0.98	0.98	0.22	0.97	0.23	0.97	0.25	0.96	0.27
(f+c)=1.25	1.22	0.27	1.21	0.29	1.21	0.31	1.20	0.34

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	16°		17°		18°		19°	
	Hor. Dist.*	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev
0	92.40	26.50	91.45	27.96	90.45	29.39	89.40	30.78
2	92.37	26.55	91.42	28.01	90.42	29.44	89.36	30.83
4	92.34	26.59	91.39	28.06	90.38	29.48	89.33	30.87
6	92.31	26.64	91.35	28.10	90.35	29.53	89.29	30.92
8	92.28	26.69	91.32	28.15	90.31	29.58	89.26	30.97
10	92.25	26.74	91.29	28.20	90.28	29.62	89.22	31.01
12	92.22	26.79	91.26	28.25	90.24	29.67	89.18	31.06
14	92.19	26.84	91.22	28.30	90.21	29.72	89.15	31.10
16	92.15	26.89	91.19	28.34	90.18	29.76	89.11	31.15
18	92.12	26.94	91.16	28.39	90.14	29.81	89.08	31.19
20	92.09	26.99	91.12	28.44	90.11	29.86	89.04	31.24
22	92.06	27.04	91.09	28.49	90.07	29.90	89.00	31.28
24	92.03	27.09	91.06	28.54	90.04	29.95	88.96	31.33
26	92.00	27.13	91.02	28.58	90.00	30.00	88.93	31.38
28	91.97	27.18	90.99	28.63	89.97	30.04	88.89	31.42
30	91.93	27.23	90.96	28.68	89.93	20.09	88.86	31.47
32	91.90	27.28	90.92	28.73	89.90	30.14	88.82	31.51
34	91.87	27.33	90.89	28.77	89.86	30.19	88.78	31.56
36	91.84	27.38	90.86	28.82	89.83	30.23	88.75	31.60
38	91.81	27.43	90.82	28.87	89.79	30.28	88.71	31.65
40	91.77	27.48	90.79	28.92	89.76	30.32	88.67	31.69
42	91.74	27.52	90.76	28.96	89.72	30.37	88.64	31.74
44	91.71	27.57	90.72	29.01	89.69	30.41	88.60	31.78
46	91.68	27.62	90.69	29.06	89.65	30.46	88.56	31.83
48	91.65	27.67	90.66	29.11	89.61	30.51	88.53	31.87
50	91.61	27.72	90.62	29.15	89.58	30.55	88.49	31.92
52	91.58	27.77	90.59	29.20	89.54	30.60	88.45	31.96
54	91.55	27.81	90.55	29.25	89.51	30.65	88.41	32.01
56	91.52	27.86	90.52	29.30	89.47	30.69	88.38	32.05
58	91.48	27.91	90.48	29.34	89.44	30.74	88.34	32.09
60	91.45	27.96	90.45	29.39	89.40	30.78	88.30	32.14
(f+c)=0.75	0.72	0.21	0.72	0.23	0.71	0.24	0.71	0.25
(f+c)=1.00	0.86	0.28	0.95	0.30	0.95	0.32	0.94	0.33
(f+c)=1.25	1.20	0.35	1.19	0.38	1.19	0.40	1.18	0.42

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	20°		21°		22°		23°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	88.30	32.14	87.16	33.46	85.97	34.73	84.73	35.97
2	88.26	32.18	87.12	33.50	85.93	34.77	84.69	36.01
4	88.23	32.23	87.08	33.54	85.89	34.82	84.65	36.05
6	88.19	32.27	87.04	33.59	85.85	34.86	84.61	36.09
8	88.15	32.32	87.00	33.63	85.80	34.90	84.57	36.13
10	88.11	32.36	86.96	33.67	85.76	34.94	84.52	36.17
12	88.08	32.41	86.92	33.72	85.72	34.98	84.48	36.21
14	88.04	32.45	86.88	33.76	85.68	35.02	84.44	36.25
16	88.00	32.49	86.84	33.80	85.64	35.07	84.40	36.29
18	87.96	32.54	86.80	33.84	85.60	35.11	84.35	36.33
20	87.93	32.58	86.77	33.89	85.56	35.15	84.31	36.37
22	87.89	32.63	86.73	33.93	85.52	35.19	84.27	36.41
24	87.85	32.67	86.69	33.97	85.48	35.23	84.23	36.45
26	87.81	32.72	86.65	34.01	85.44	35.27	84.18	36.49
28	87.77	32.76	86.61	34.06	85.40	35.31	84.14	36.53
30	87.74	32.80	86.57	34.10	85.36	35.36	84.10	36.57
32	87.70	32.85	86.53	34.14	85.31	35.40	84.06	36.61
34	87.66	32.89	86.49	34.18	85.27	35.44	84.01	36.65
36	87.62	32.93	86.45	34.23	85.23	35.48	83.97	36.69
38	87.58	32.98	86.41	34.27	85.19	35.52	83.93	36.73
40	87.54	33.02	86.37	34.31	85.15	35.56	83.89	36.77
42	87.51	33.07	86.33	34.35	85.11	35.60	83.84	36.80
44	87.47	33.11	86.29	34.40	85.07	35.64	83.80	36.84
46	87.43	33.15	86.25	34.44	85.02	35.68	83.76	36.88
48	87.39	33.20	86.21	34.48	84.98	35.72	83.72	36.92
50	87.35	33.24	86.17	34.52	84.94	35.76	83.67	36.96
52	87.31	33.28	86.13	34.57	84.90	35.80	83.63	37.00
54	87.27	33.33	86.09	34.61	84.86	35.85	83.59	37.04
56	87.24	33.37	86.05	34.65	84.82	35.89	83.54	37.08
58	87.20	33.41	86.01	34.69	84.77	35.93	83.50	37.12
60	87.16	33.46	85.97	34.73	84.73	35.97	83.46	37.16
(f+c)=0.75	0.70	0.26	0.70	0.27	0.69	0.29	0.69	0.30
(f+c)=1.00	0.94	0.35	0.93	0.37	0.92	0.38	0.92	0.40
(f+c)=1.25	1.17	0.44	1.16	0.46	1.15	0.48	1.15	0.50

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

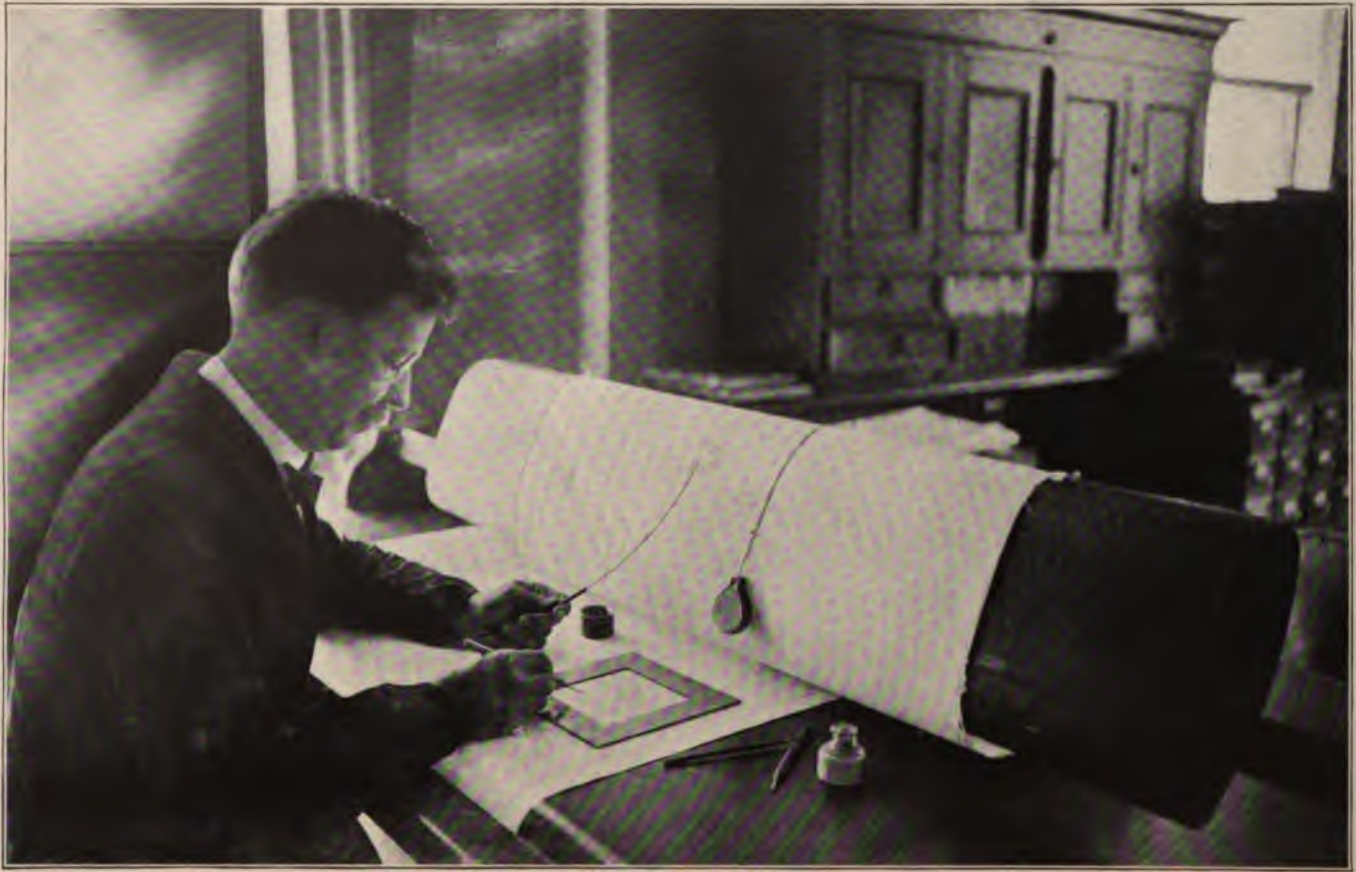
Minutes	24°		25°		26°		27°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	83.46	37.16	82.14	38.30	80.78	39.40	79.39	40.45
2	83.41	37.20	82.09	38.34	80.74	39.44	79.34	40.49
4	83.37	37.23	82.05	38.38	80.69	39.47	79.30	40.52
6	83.33	37.27	82.01	38.41	80.65	39.51	79.25	40.55
8	83.28	37.31	81.96	38.45	80.60	39.54	79.20	40.59
10	83.24	37.35	81.92	38.49	80.55	39.58	79.15	40.62
12	83.20	37.39	81.87	38.53	80.51	39.61	79.11	40.66
14	83.15	37.43	81.83	38.56	80.64	39.65	79.06	40.69
16	83.11	37.47	81.78	38.60	80.41	39.69	79.01	40.72
18	83.07	37.51	81.74	38.64	80.37	39.72	78.96	40.76
20	83.02	37.54	81.69	38.67	80.32	39.76	78.92	40.79
22	82.98	37.58	81.65	38.71	80.28	39.79	78.87	40.82
24	82.93	37.62	81.60	38.75	80.23	39.83	78.82	40.86
26	82.89	37.66	81.56	38.78	80.18	39.86	78.77	40.89
28	82.85	37.70	81.51	38.62	80.14	39.90	78.73	40.92
30	82.80	37.74	81.47	38.86	80.09	39.93	78.68	40.96
32	82.76	37.77	81.42	38.89	80.04	39.97	78.63	40.99
34	82.72	37.81	81.38	38.93	80.00	40.00	78.58	41.02
36	82.67	37.85	81.33	38.97	79.95	40.04	78.54	41.06
38	82.63	37.89	81.28	39.00	79.90	40.07	78.49	41.09
40	82.58	37.93	81.24	39.04	79.86	40.11	78.44	41.12
42	82.54	37.96	81.19	39.08	79.81	40.14	78.38	41.16
44	82.49	38.00	81.15	39.11	79.76	40.18	78.34	41.19
46	82.45	38.04	81.10	39.15	79.72	40.21	78.30	41.22
48	82.41	38.08	81.06	39.18	79.67	40.24	78.25	41.26
50	82.36	38.11	81.01	39.22	79.62	40.28	78.20	41.29
52	82.32	38.15	80.97	39.26	79.58	40.31	78.15	41.32
54	82.27	38.19	80.92	39.29	79.53	40.35	79.10	41.35
56	82.23	38.23	80.87	39.33	79.48	40.38	79.06	41.39
58	82.18	38.26	80.83	39.36	76.44	40.42	78.01	41.42
60	82.14	38.30	80.78	39.40	79.39	40.45	77.96	41.45
(f+c)=0.75	0.68	0.31	0.68	0.32	0.67	0.33	0.66	0.35
(f+c)=1.00	0.91	0.41	0.90	0.43	0.89	0.45	0.89	0.46
(f+c)=1.25	1.14	0.52	1.13	0.54	1.12	0.56	1.11	0.58

TABLE I.

(Continued.)

Horizontal Distances and Elevations from Stadia Readings.

Minutes	28°		29°		30°	
	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.	Hor. Dist.	Diff. Elev.
0	77.96	41.45	76.50	42.40	75.00	43.30
2	77.91	41.48	76.45	42.43	74.95	43.33
4	77.86	41.52	76.40	42.46	74.90	43.36
6	77.81	41.55	76.35	42.49	74.85	43.39
8	77.77	41.58	76.30	42.53	74.80	43.42
10	77.72	41.61	76.25	42.56	74.75	43.45
12	77.67	41.65	76.20	42.59	74.70	43.47
14	77.62	41.68	76.15	42.62	74.65	43.50
16	77.57	41.71	76.10	42.65	74.60	43.53
18	77.52	41.74	76.05	42.68	74.55	43.56
20	77.48	41.77	76.00	42.71	74.49	43.59
22	77.42	41.81	75.95	42.74	74.44	43.62
24	77.38	41.84	75.90	42.77	74.39	43.65
26	77.33	41.87	75.85	42.80	74.34	43.67
28	77.28	41.90	75.80	42.83	74.29	43.70
30	77.23	41.93	75.75	42.86	74.24	43.73
32	77.18	41.97	75.70	42.89	74.19	43.76
34	77.13	42.00	75.65	42.92	74.14	43.79
36	77.09	42.03	75.60	42.95	74.09	43.82
38	77.04	42.06	75.55	42.98	74.04	43.84
40	76.99	42.09	75.50	43.01	73.99	43.87
42	76.94	42.12	75.45	43.04	73.93	43.90
44	76.89	42.15	75.40	43.07	73.88	43.93
46	76.84	42.19	75.35	43.10	73.83	43.95
48	76.79	42.22	75.30	43.13	73.78	43.98
50	76.74	42.25	75.25	43.16	73.73	44.01
52	76.69	42.28	75.20	43.18	73.68	44.04
54	76.64	42.31	75.15	43.21	73.63	44.07
56	76.59	42.34	75.10	43.24	73.58	44.09
58	76.55	42.37	75.05	43.27	73.52	44.12
60	76.50	42.40	75.00	43.30	73.47	44.15
(f+c)=0.75	0.66	0.36	0.65	0.37	0.65	0.38
(f+c)=1.00	0.88	0.48	0.87	0.49	0.86	0.51
(f+c)=1.25	1.10	0.60	1.09	0.62	1.08	0.64



ONE STAGE IN THE MAKING OF GOVERNMENT CHARTS

Reducing from field plane-table chart by method of least squares. Hair frame lying flat on table; transparent frame held onto sheet by weighted cord; and pointer in position to keep the place for the draftsman.

TABLE II. Dimensions and Co-ordinates for Map Construction.

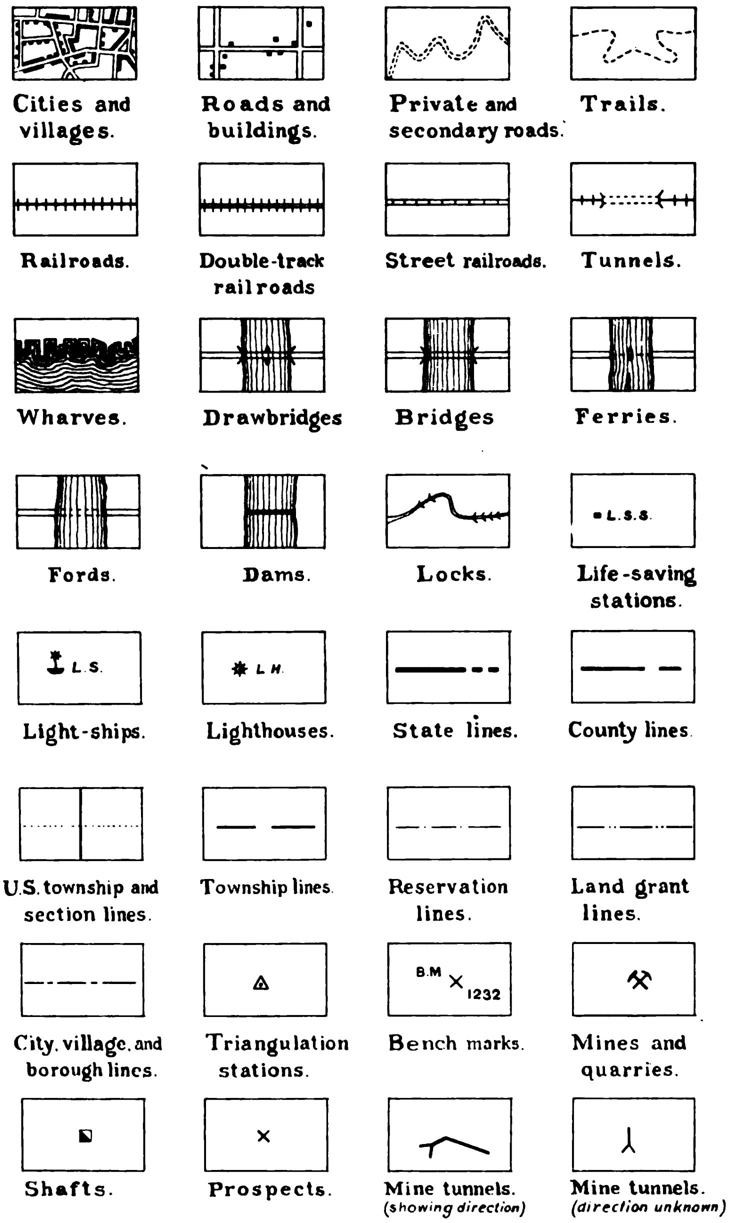
Latitude	Length of 1° measured on a parallel in statute miles	Length of 1° measured on a meridian in statute miles	Length of side of tangent cone in statute miles	MAP CO-ORDINATES—IN METERS													
				0° 15′ from center		0° 30′ from center		0° 45′ from center		1° from center		1° 30′ from center		2° from center		3° from center	
				X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
38°	54.579	68.969	5079	21959	29	43917	118	65875	265	87833	472	131746	1062	175656	1888	263458	4247
39°	53.829	68.981	4901	21657	30	43314	119	64971	268	86627	476	129937	1070	173243	1903	259839	4281
40°	53.063	68.993	4730	21349	30	42698	120	64046	269	85394	479	128088	1078	170778	1916	256140	4311
41°	52.281	69.006	4566	21034	30	42068	120	63102	271	84136	482	126200	1084	168260	1927	252363	4335
42°	51.483	69.018	4408	20713	30	41426	121	62139	272	82851	484	124273	1088	165691	1935	248508	4354
43°	50.669	69.030	4257	20386	30	40771	121	61157	273	81541	485	122308	1092	163071	1941	244578	4367
44°	49.840	69.042	4111	20052	30	40104	121	60156	273	80206	486	120306	1094	160401	1945	240572	4375
45°	48.995	69.054	3970	19712	30	39424	122	59136	274	78847	486	118267	1095	157682	1946	236493	4378
46°	48.136	69.066	3834	19366	30	38733	122	58098	273	77464	486	116192	1094	154915	1945	232342	4376
47°	47.261	69.079	3703	19015	30	38029	121	57043	273	76056	485	114080	1092	152100	1942	228119	4368
48°	46.372	69.091	3575	18657	30	37314	121	55970	272	74626	484	111934	1089	149239	1936	223827	4355
49°	45.469	69.103	3452	18294	30	36587	120	54880	271	73172	482	109754	1084	146331	1928	219465	4337
50°	44.552	69.115	3332	17924	30	35849	120	53773	270	71696	479	107540	1078	143379	1917	215037	4313

TABLE II. Dimensions and Co-ordinates for Map Construction.

Latitude	Length of 1° measured on a parallel in statute miles	Length of 1° measured on a meridian in statute miles	Length of side of tangent cone in statute miles	MAP CO-ORDINATES—IN METERS													
				0° 15′ from center		0° 30′ from center		0° 45′ from center		1° from center		1° 30′ from center		2° from center		3° from center	
				X	Y	X	Y	X	Y	X	Y	X	Y	X	Y	X	Y
25°	62.729	68.829	8504	25238	23	50476	93	75713	209	100951	372	151425	838	201896	1489	302831	3351
26°	62.212	68.839	8131	25030	24	50060	96	75089	215	100118	383	150176	862	200231	1532	300332	3447
27°	61.676	68.848	7784	24814	25	49628	98	74442	221	99256	393	148882	885	198505	1573	297742	3539
28°	61.122	68.858	7459	24591	25	49182	101	73772	227	98363	403	147542	907	196719	1612	295062	3627
29°	60.548	68.869	7156	24360	26	48720	103	73080	232	97440	412	146157	928	194872	1649	292291	3710
30°	59.956	68.879	6870	24122	26	48244	105	72366	237	96487	421	144728	947	192967	1684	289432	3789
31°	59.345	68.890	6602	23876	27	47753	107	71629	241	95505	429	143255	966	191002	1717	286484	3863
32°	58.716	68.901	6349	23624	27	47247	109	70871	246	94494	437	141738	983	188980	1748	283449	3933
33°	58.071	68.912	6109	23364	28	46727	111	70091	250	93454	444	140178	999	186899	1777	280328	3997
34°	57.407	68.923	5882	23097	28	46193	113	69289	254	92385	451	138575	1014	184762	1803	277121	4057
35°	56.725	68.935	5666	22822	29	45645	114	68467	257	91289	457	136930	1028	182568	1828	273830	4112
36°	56.027	68.946	5461	22541	29	45083	116	67624	260	90164	462	135243	1041	180319	1854	270455	4162
37°	55.311	68.958	5266	22253	29	44507	117	66760	263	89012	467	133515	1052	178015	1870	266997	4207

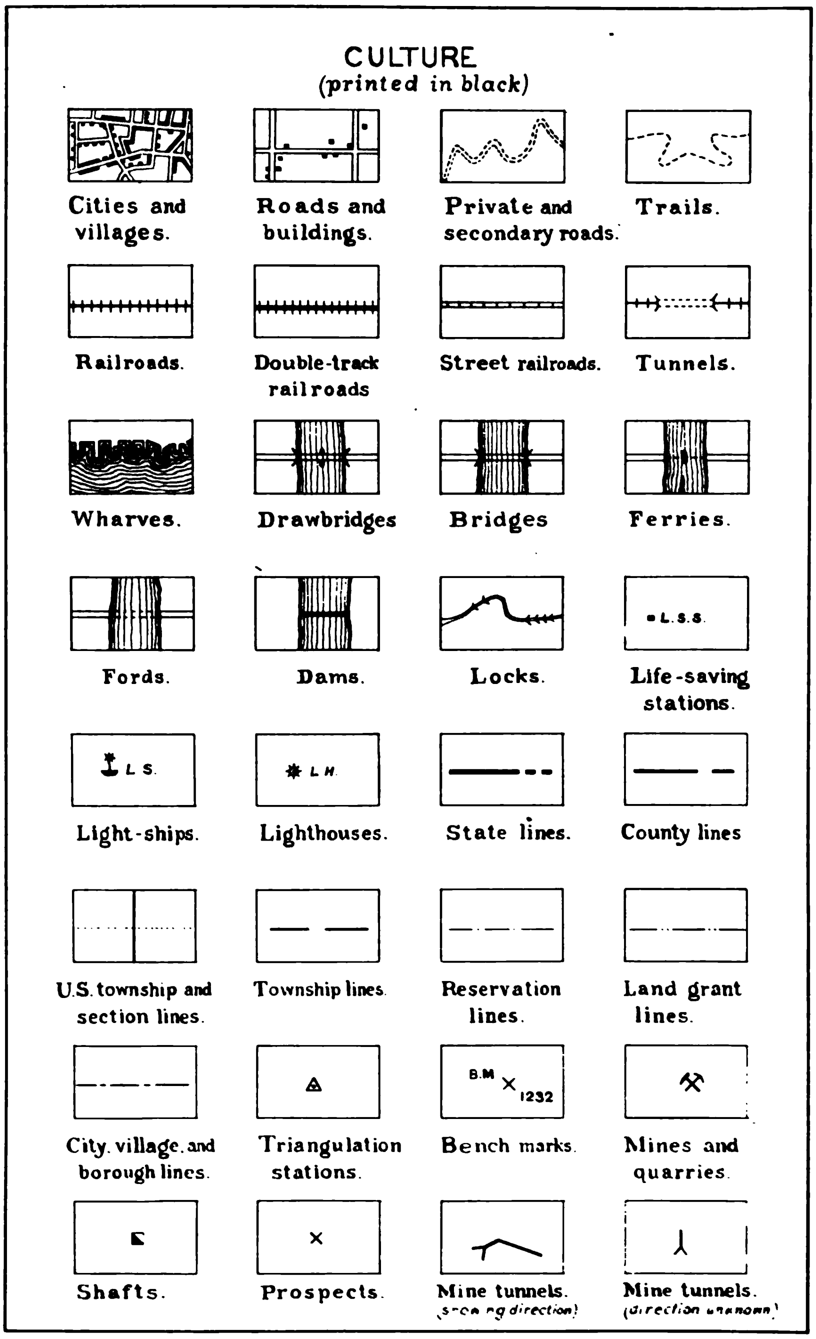
CONVENTIONAL SIGNS. PLATE I.

CULTURE
(printed in black)



For illustrations of many of these signs in their true colors, see Plate III.

CONVENTIONAL SIGNS. PLATE I.



For illustrations of many of these signs in their true colors, see Plate III.

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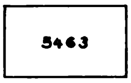
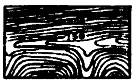
CONVENTIONAL SIGNS PLATE II

RELIEF

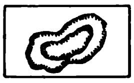
(printed in brown)



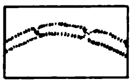
Figures.
(showing heights above mean sea level instrumentally determined)



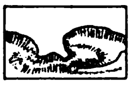
Contours
(showing height above sea, horizontal form, and steepness of slope of the surface.)



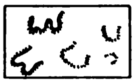
Depression contours.



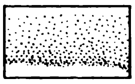
Levees.



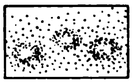
Cliffs.



Mine dumps.



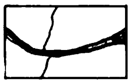
Sand.



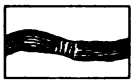
Sand dunes.

WATER

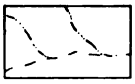
(printed in blue)



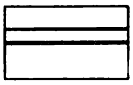
Streams.



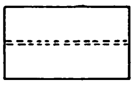
Falls and rapids.



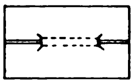
Intermittent streams.



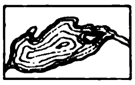
Canals and ditches.



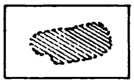
Aqueducts.



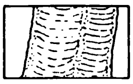
Aqueduct tunnels.



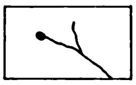
Lakes and ponds.



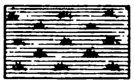
Intermittent lakes.



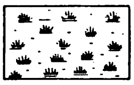
Glaciers.



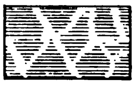
Springs.



Salt marshes.



Fresh marshes.



Tidal flats.

For illustrations of many of these signs in their true colors, see Plate III.

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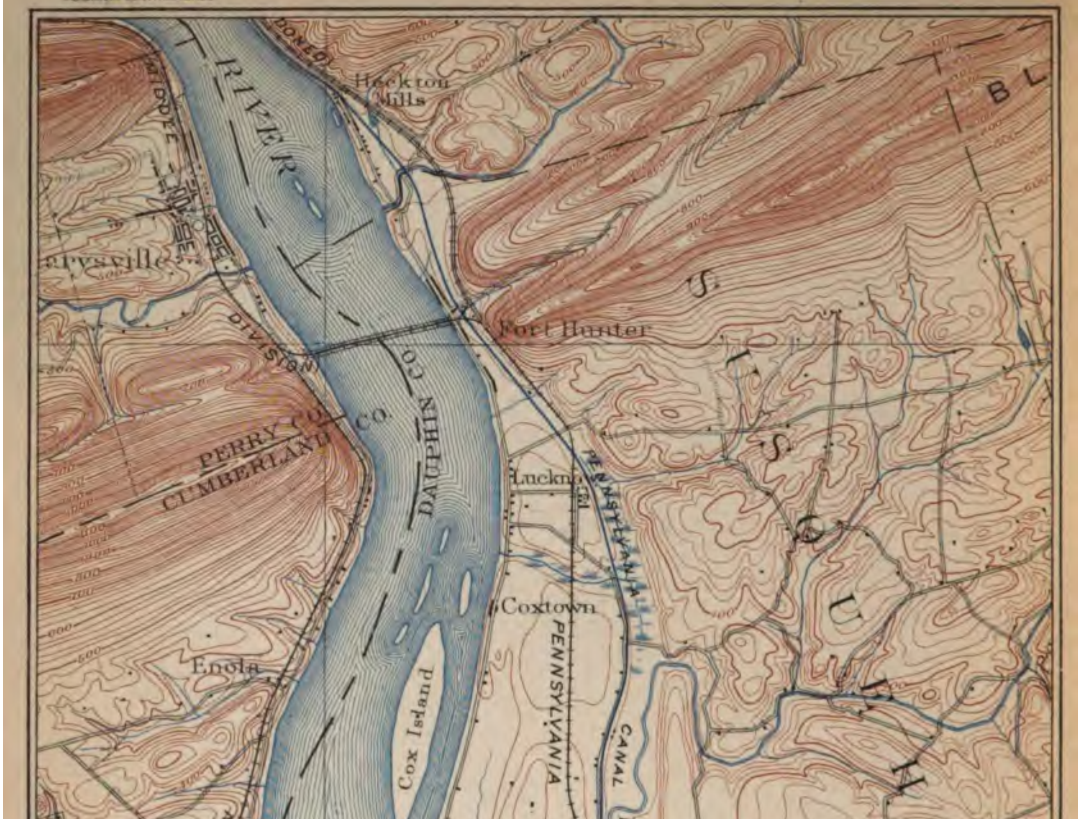

PLATE III.—Part of Harrisburg, Pa., sheet. U. S. Geological Survey. Scale 1:62500

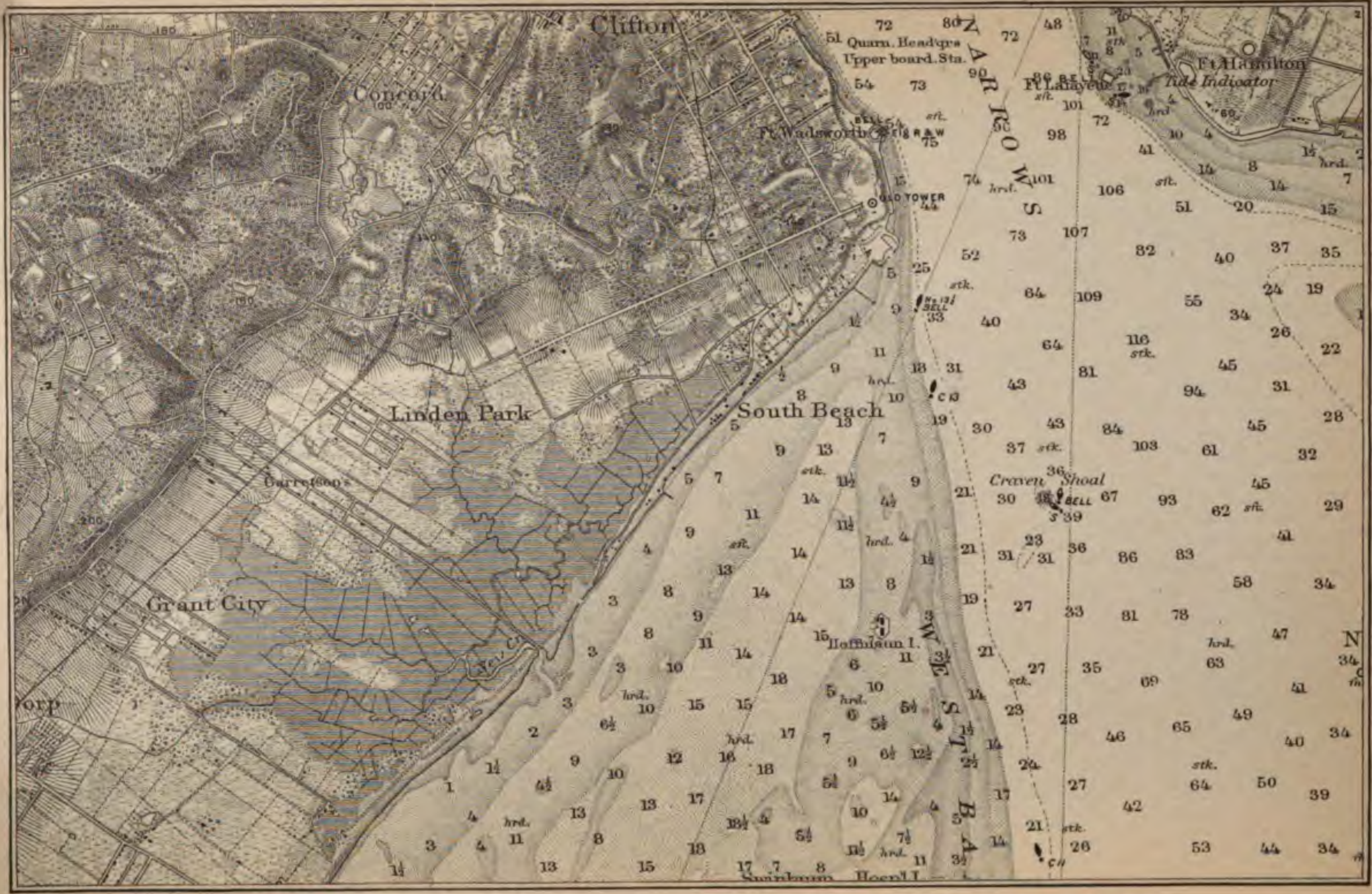

PLATE IV.—Part of New York Harbor. Relief shown by hachures. U. S. Coast Survey. Scale 1:40000

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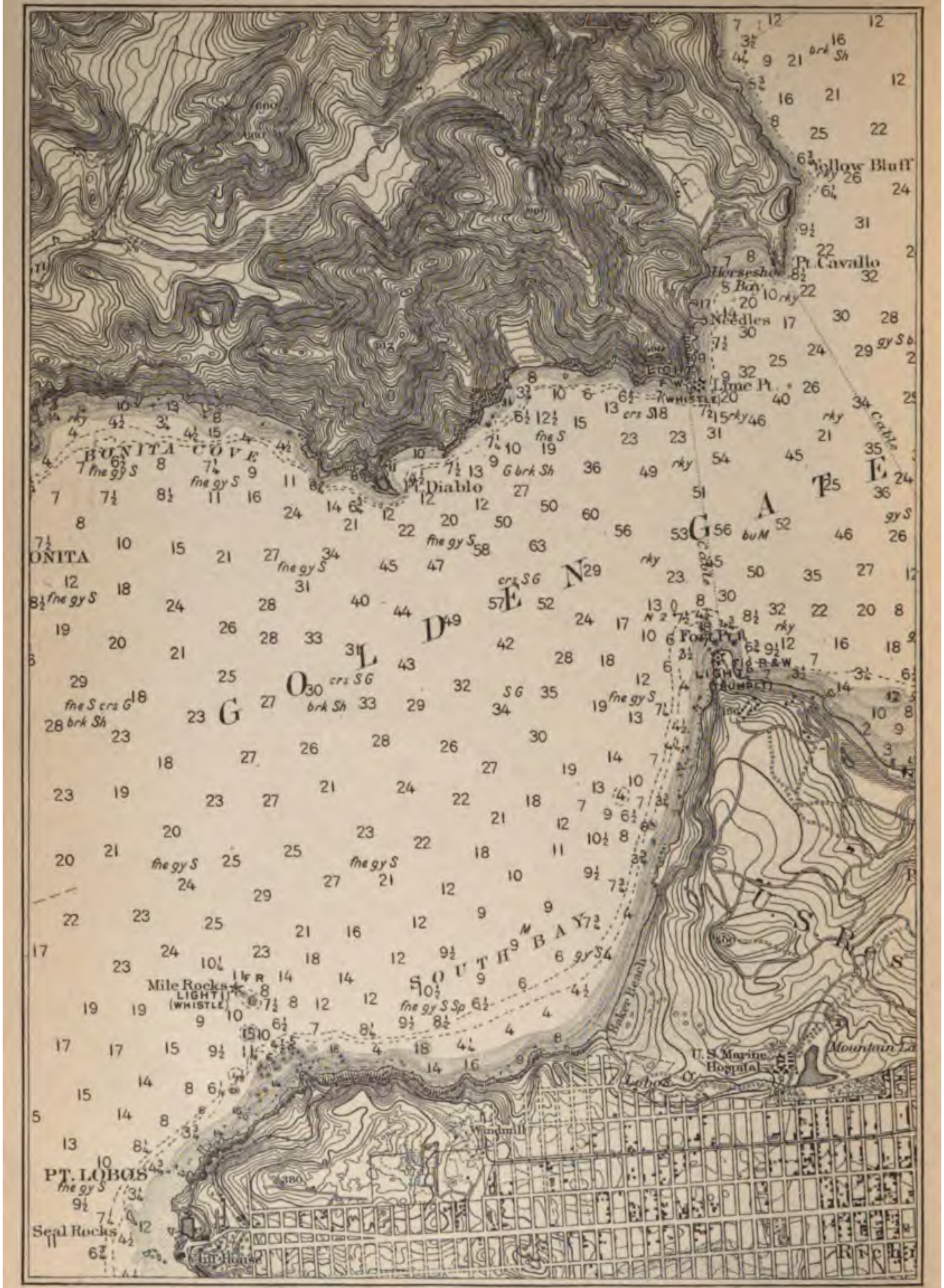
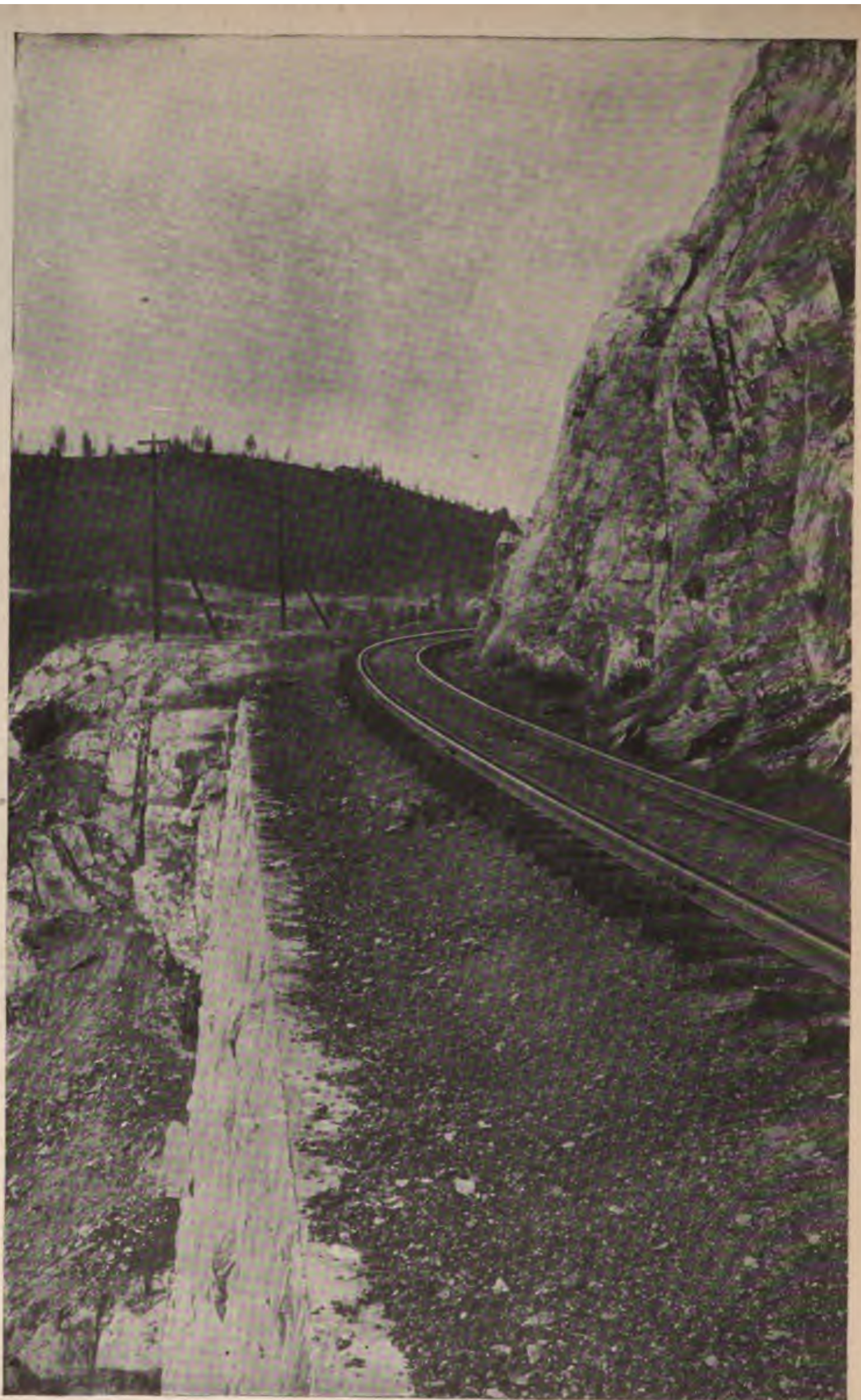


PLATE V.—Part of San Francisco Entrance. Relief shown by contours. U. S. Coast Survey. Scale 1:40000



A RAILROAD CURVE IN THE SIERRA NEVADA, CALIFORNIA

View on the line of the Southern Pacific Railway, where the road rounds Cape Horn. In *mountainous regions*, the construction of such a curve and embankment as here shown presupposes *the most* careful preliminary work by surveying parties, who, in fact, are the *pioneers without whose* work such construction would be impossible.

RAILROAD ENGINEERING.

PART I.

RAILROAD SURVEYS.

1. General Principles. The engineer should have first, a thorough appreciation of the objects to be accomplished by the surveys. He should realize that, except in the rare cases where it is difficult to find *any* practicable line, very little engineering training or ability is required to lay out a line over which it would be physically possible to run trains. A line as laid out may violate all rules of location, may be expensive to operate and have disadvantages which will discourage traffic, and yet trains *can* be run over it. From the infinite number of possible locations, the engineer must select the location which best satisfies the various conflicting interests. His value as an engineer depends on his ability to interpret the natural conditions and design the line accordingly. This ability is only obtained by a thorough knowledge of the whole subject of railroad engineering, supplemented by practical experience. It is therefore true that many of the following statements will not be thoroughly appreciated until the student has covered the whole subject and then reviews it.

2. Conflicting Interests. There are several classes of interests, which are generally more or less conflicting, which affect the location of every line·

(*a*) *The initial cost should be a minimum*, but the cheapest road generally has sharp curvature, steep grades and inconvenient location.

(*b*) *The operating expenses per train mile should be a minimum*, which is generally equivalent to saying that the curvature should be light and the grades low, but this is usually unobtainable except at great cost.

(*c*) *The location should be convenient to sources of traffic* so that the maximum traffic will be obtained, but this is generally very costly.

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A little study will show the frequent conflict of the above conditions. When a proposed location evidently combines the above interests advantageously, instead of bringing them into conflict, then there is no doubt as to the proper location, unless it affects unduly the adjacent location. The best engineering ability is a cheap investment when deciding on a location which requires a delicate balancing of the claims of several possible routes, each with its own combination of greater or less initial cost, greater or less operating expenses, and greater or less effect on the probable revenue of the road.

RECONNOISSANCE SURVEYS.

3. **Essential Problem.** From the above considerations it may readily be seen that the first survey to be made (called the reconnoissance survey) consists essentially of a broad examination of the country through which the road is expected to pass. Business considerations usually predetermine that the road is to connect certain termini and also pass through certain intermediate important towns or cities, but the problem consists in finding the best route between the predetermined points. When two consecutive predetermined points lie in the same valley or on the same bank of a river too large to be easily bridged, the location is self-evident. If the river is smaller, easily bridged, has sharp bends, with variable banks and important towns on either bank, it will usually require a close examination of each bank to determine where to cross if at all. When the two points are many miles apart, lie in different valleys, and are therefore separated by one or more summits, the selection of the best route becomes more and more complicated as the number of possible routes becomes greater. It is generally true, although not invariably, that a cross-country route which includes the lowest summits and the highest *low points* (such as river crossings) will give the best grades. Since the "ruling grade" is the most important physical consideration for the engineer, as will be developed later, the chief work of the reconnoissance survey (apart from considerations of probable traffic) is the determination of the elevations of summits and sags and the distance between them, together with the constructive character of the country.

4. Utilization of Existing Maps. The U. S. Geological Survey has already published contour maps of a large part of the country which enable an engineer to select a line with even greater ease and certainty than he can from a reconnoissance map made for the purpose (as usually made), since the U. S. G. S. maps show the *whole* country and enable the engineer to rapidly compare a dozen suggested routes instead of confining his attention to the (usually) limited area of the special map. The errors of the U. S. G. S. maps will seldom if ever be sufficient to vitiate the accuracy of the preliminary route laid out from them. Usually a brief study of the map will demonstrate that one (or perhaps two or three) general route has advantages so pronounced over all other possible routes that the choice is immediately made or is at least reduced to the comparison of two or three lines which are so nearly equal that closer and more detailed surveys are necessary to decide between them. County atlases are usually sufficiently accurate for reconnoissance purposes to the extent of giving the relative horizontal positions of governing points of the survey. Elevations may be determined (as described later) and plotted on these maps.

5. Surveying Methods. When reliable contour maps are unavailable, some of the following methods may be used to fill out existing maps or to make a complete reconnoissance survey. The essential point is the rapid determination of those details from which one route is shown to be superior to another. Nothing useless should be surveyed and no time should be wasted on an unnecessary degree of accuracy. The physical characteristics of two routes have usually such differences that they are apparent even with rapid and approximate methods of surveying. If two routes are so nearly equal that a decisive choice cannot be made from the results of reconnoissance surveys, it shows that a more accurate survey should be made of both routes.

6. Elements. The three elements of the survey of any line are (*a*) the length, (*b*) the direction, and (*c*) the slope or the relative elevation of the two ends. *Distance.* The length is sometimes determined with sufficient accuracy by pacing, the steps being counted with a pedometer. In an open prairie country, where a buggy may be run, an odometer attached to a wheel will count the revolutions. An odometer on a wheel, attached to a frame and trundled like a wheelbarrow, has been used for the same purpose. A large telescope, mounted with a universal joint on a very light tripod, and fitted with stadia wires so adjusted that distances of 2,000 or even 2,500 feet can be read to the nearest 10 feet on a 10-foot rod, will give the distances between widely separated stations with sufficient accuracy and extreme rapidity. *Direction* may be obtained with sufficient accuracy with a compass—even of the pocket type. *Leveling.* Spirit leveling is too slow and expensive for the rapid surveying here required. If stadia methods are used with an instrument provided for reading vertical angles, the inclination of all lines may be observed and the elevations of all stations computed. A still more rapid method of observing differences of elevation with sufficient accuracy for the purpose is found in the use of an aneroid barometer, supplemented by another aneroid or preferably by a mercurial barometer. The mercurial, or the office aneroid, is kept at some office whose elevation is known and observations are regularly taken (say every half hour) during the period when observations are being taken in the field with the field aneroid. The field aneroid is taken to each place, within a range of several miles, where elevations are desired. At each point there should be noted (see the form of notes below) the time, the described location, the aneroid reading and the temperature. If possible, duplicate readings should be taken on the trip to and from the office on all important points. The elevations of succeeding office locations made, may be determined with the field aneroid if necessary, but of course extra care should be taken with such work.

Aneroids are usually "compensated for temperature," *i.e.*, so adjusted that they will give a true reading regardless of temperature. If an aneroid has not been so adjusted, it should be carefully compared with a standard mercurial barometer under widely varying conditions of temperature and a tabular form should be made out for that aneroid showing the correction to be applied at any given temperature. On account of the expansion of mercury with temperature, and also the expansion (at a different rate) of the tube and cistern, all readings of the mercurial barometer must be "reduced to 32° F.," *i.e.*, reduced to the reading it would have, if the temperature of the instrument were 32° F. This is readily accomplished by means of Table XI.* At the office, each half-hourly observation should include the time, the reading of the scale showing the height of the mercury, the reading of the "attached thermometer" (the thermometer attached to the mercurial) and also the temperature of the external air. When the mercurial is indoors these two temperatures may differ somewhat. When reducing the observations interpolation should be made if necessary between the reduced office observations to determine the probable reading of the mercurial at the time of any given field observation. Determine from Table XII the heights corresponding to the field reading and reduced office reading for each pair of observations. Their difference is the *approximate* difference of elevation of the office and of the place of the field observation. If necessary this may be corrected by an amount equal to the approximate difference of elevation times a coefficient derived from Table XIII. This coefficient is found opposite the number which gives the *sum* of the temperatures in the field and outside the office. The correction is frequently too small to be noticed. An approximate calculation will often show this, or will give a solution to the nearest foot, which is amply accurate. An aneroid, no matter how perfect, will seldom agree exactly with a mercurial barometer, and even if adjusted to the same reading will soon indicate some discrepancy. It is therefore better to leave the adjustment undisturbed and apply corrections. The aneroid should therefore be compared with the mercurial before leaving headquarters for a day's work, and the readings of both and their *difference* should be recorded. Immediately after returning from the day's work the aneroid should again be compared. The absolute reading of the mercurial will probably be higher or lower, but the *difference* should be nearly the same, although it is found that an aneroid will lag somewhat behind its true reading, especially if it has been subjected to an extreme variation of pressure. All the field readings of the aneroid should therefore be corrected by the *mean* of the initial and final differences. The method and the above explanation may be illustrated by the following numerical examples:

7. Examples. 1. Given a reading of 28.692 on a mercurial barometer, what is its reading when reduced to 32° F., the reading of the attached thermometer being 68.5° F.? In Table XI, under 28.5 and opposite 68°, we find – .101. Under 29.0 and for 68° we find – .103. For 28.692 and 68° it evidently should be – .102 (to the nearest thousandth). Similarly for 28.692 and 69° we may derive – .105. For 28.692 and 68.5° it would be the mean or – .1035, which we will call – .103 since it is useless to compute the correction closer than the nearest thousandth. Then since the correction is – .103, the corrected reading should be 28.589. With a little practice the interpolations, when necessary, may be made in far less time than it takes to describe it.

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* The tables referred to in this book are TRIGONOMETRIC TABLES, prepared by Walter Loring Webb, and published by the American School of Correspondence.

2. Verify the following reductions:

Bar. reading.	Temp.	Reduced reading.
26.426	58° F.	26.356
27.892	78.5	27.767
28.475	85.	28.330
30.847	48.5	30.792

3. Reduce the following readings: 27.294, 47°; 29.462, 87°; 26.230, 78.5°; 25.241, 62°; 26.481, 75°; 29.625, 89.5°; 30.942, 88.5°; 29.784, 46.5°; 28.386, 48°; 27.942, 74.5°.

4. Compute the barometric elevation corresponding to a reading of 28.589. From Table XII the reading for 28.5 is 1397 and the difference for .01 is – 9.5; therefore, for .089 the correction will be – 9.5 × 8.9 = – 84.55, or in whole numbers – 85. Then 1397 – 85 = 1312, the corrected reading.

5. Verify the following elevations from the reduced readings: 26.356, 27.767, 28.330, and 30.792; *i.e.*, 3528, 2107, 1560, and – 710.

6. Compute the barometric elevations corresponding to the *reduced readings* found by solving Example 3.

7. With an approximate difference of elevation of – 136 feet and field and office temperatures of 62° and 67°, what is the true difference of elevation? 62 + 67 = 129. For 129° the coefficient is (by interpolation) + .0357. 136 × (+ .0357) = + 4.8552. For this slight difference of elevation, the coefficient is *far* more accurate than necessary, and of course the correction is called + 5.

The difference of elevation should be *increased* by 5, but the difference is essentially negative. Therefore we have as the correction − (+ 5). The true difference of elevation is − 136 − (+ 5) = − 141.

8. The following example shows not only the method of recording the observations but also the complete solution of a problem.

Time.	Mercurial barometer.	Attached thermometer.	Reduction to 32° F.	Corrected reading.	External thermometer.
7:00 A. M.	28.692	62°	−.087	28.605	60° F.
:30	.724	64	−.092	.632	62
8:00	.756	66.5	−.099	.657	64
:30	.782	68	−.102	.680	65
9:00	.824	69	−.105	.719	66

The observations taken in the field at this time were as given in the first four columns of the following tabular form. The other columns are computed later in the office.

(Left-hand page of notes.)

Time.	Place.	Aneroid.	Therm.	Corrected aneroid.	Corrected mercurial.
7:00 A. M.	Office..........	28.743	62°	....	28.605
7:20	R. R. Junction.	.769	63	28.631	.623
8:10	Blue River	.860	65	.722	.665
8:50	Saddle in Beanpole ridge	.522	66	.384	.706

(Right-hand page of notes.)

Ext. temp. office.	Approx. field reading.	Approx. office reading.	Difference.	Correction for temp.	Difference of elevation.
....	......	......	......	......	......
62°	1273	1280	− 7	0	− 7
64	1186	1240	− 54	− (+ 2)	− 56
66	1508	1201	+ 307	+ 12	+ 319

8. **Low Ruling Grades.** It will be developed later that a low ruling grade is of prime importance. The approximate value of the ruling grade is determined from the reconnoissance survey. If the country is mountainous, it may be necessary to "develop" the line in order to reduce the grade. "Development" here

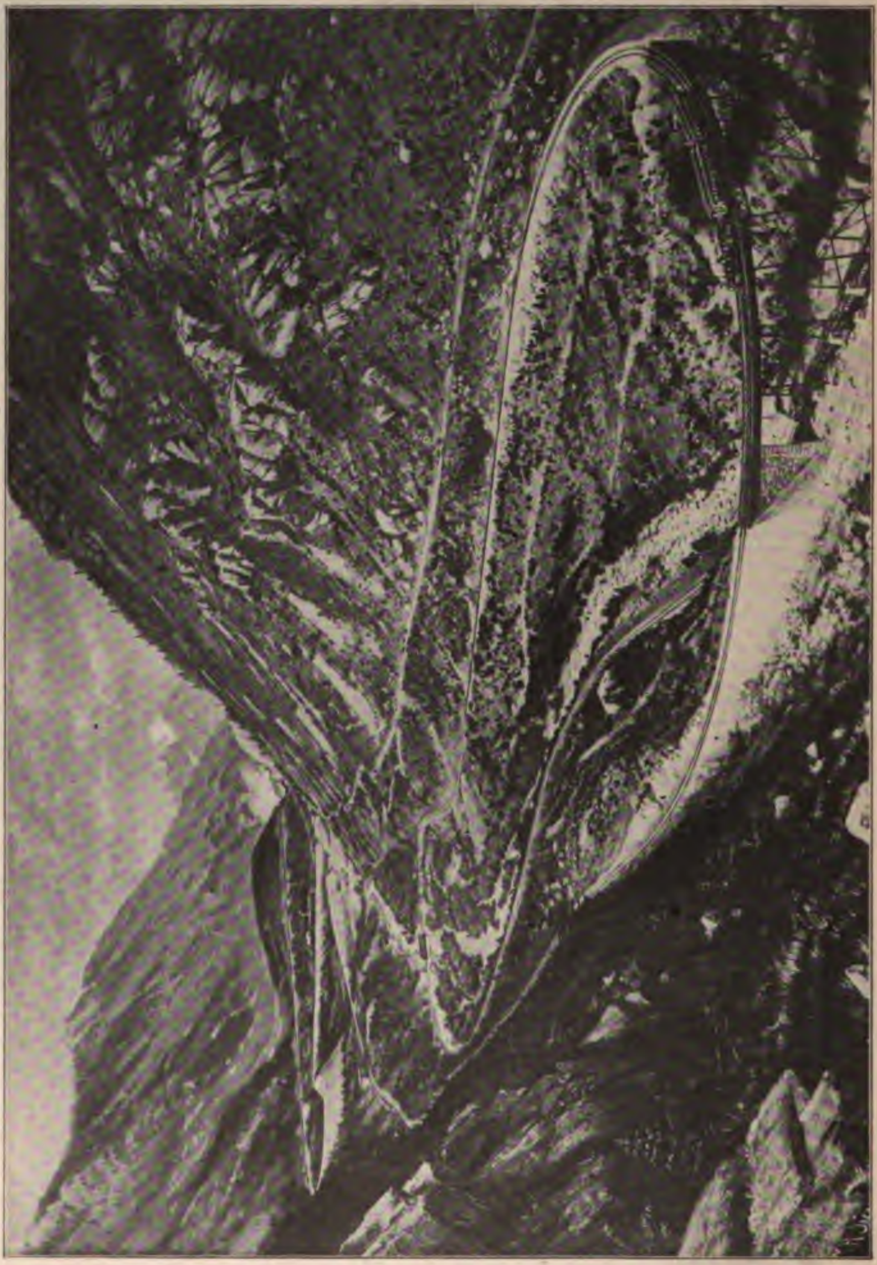


Fig. 1. Georgetown Spiral.

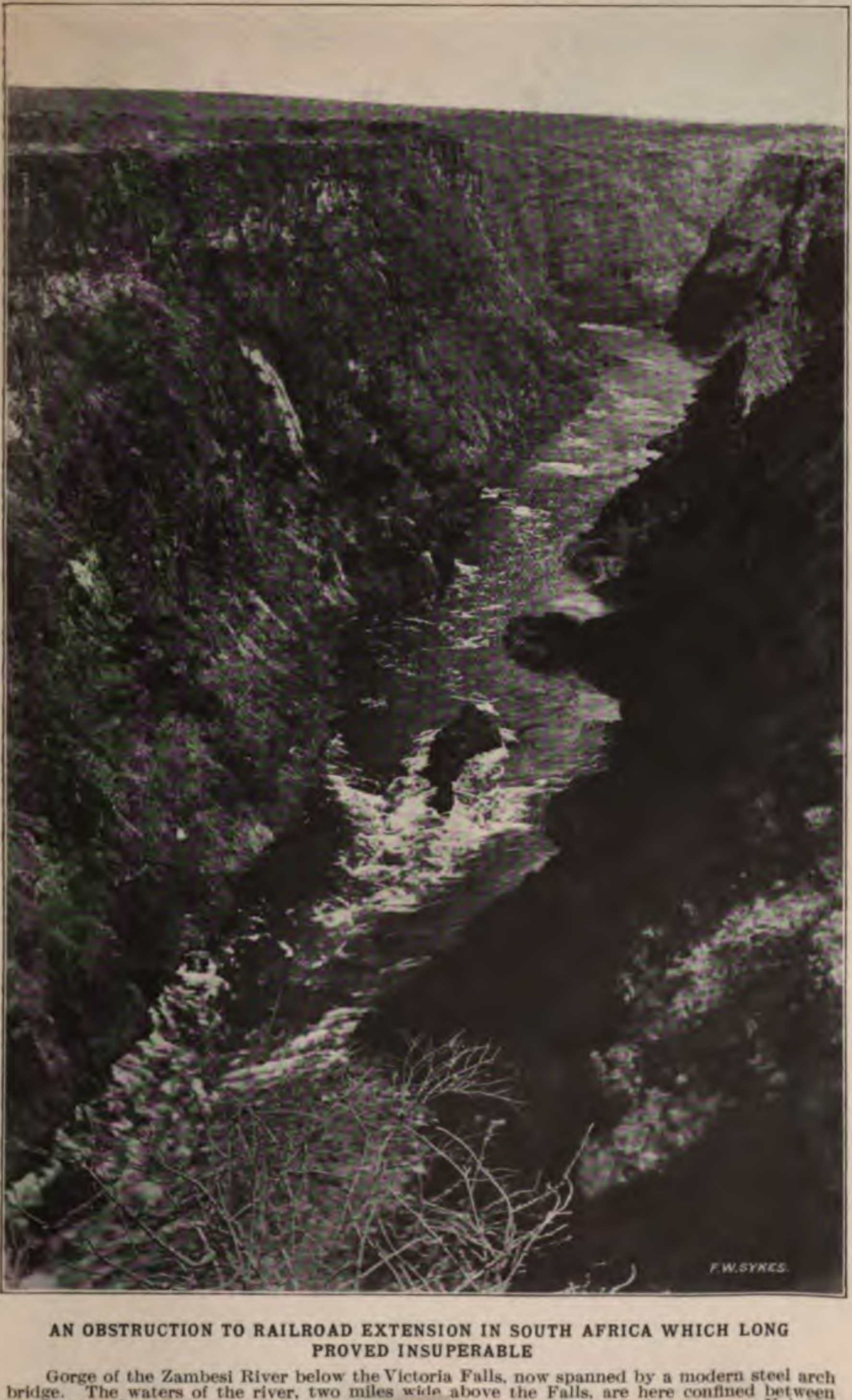
means a deliberate increase in the length of the line between two predetermined points so that the rate of grade shall be as low as desired. The Georgetown spiral, shown in Fig. 1, is perhaps the most famous example in this country of this general method. A study of the course of the track will illustrate several methods of taking advantage of the topography and attaining a considerable elevation although the grade is kept low.

PRELIMINARY SURVEYS.

9. General Object. The reconnoissance survey has shown that the best location for the road will lie somewhere through a certain belt of country. In some places this belt may be very narrow, *i.e.*, certain topographical features will determine that the road *must* pass through a strip but little if any wider than the roadbed requirements. In other places the choice of possible location is so widened that it is necessary to survey everything within reach of the backbone line of the survey. The willingness or financial ability of the company to ignore minor topographical considerations and incur heavy expense in order to obtain economic advantages, may also widen the area of possible location. As a general statement, the width of the belt surveyed should so vary as to include all practicable locations along that general route.

10. Cross Section Method. A broken line is run which shall lie as near the expected location line as possible. The bearing and length of each segment of the broken line is determined and also all essential topographical features on either side. Bearings are sometimes taken only with a compass, which has the advantage of great rapidity but lessened accuracy. For more accurate work, true azimuth is carried along by means of back sights at previous stations. The azimuths between stations should be checked by means of needle readings. It is advisable to determine exact azimuth at the beginning of a survey and at intervals of a few miles. This may be done by observations on Polaris (see Plane Surveying, Part II, Pages 95 to 97), or still better, by solar observations which may be taken with great accuracy at any time of day. Set stakes at each even 100 feet. In general the instrument stations will not occur at the even 100-foot distances, but the odd distance should always be carried on to the next course. The stakes should be about fifteen inches long and about one-and-one-quarter inches square. Stakes with a cross-section of one inch by one-and-one-half inches are preferred by some. The stakes indicating the 100-foot stations should be driven to within five inches of the ground. Stakes indicating the locations of the transit (called hubs) should be driven flush with the ground. A "witness stake" should then be driven three feet to the right and on this stake should be marked the station number and the "plus distance"; *e.g.*, the stake might show 137 + 46, which would indicate that the stake was 46 feet beyond Sta. 137, and 13,746 feet from the starting point. Station stakes should be marked with the station number on the *rear* side of the stake. Immediately following the transit party, the level party should obtain the elevations above the datum plane of all stations and substations, ridges, sags, river banks and any point where the profile changes abruptly.

11. Cross Sectioning. Use a Locke level, resting on a five-foot stick, a 50-foot tape and a ten-foot rod graduated to feet and tenths. The cross-section party takes cross sections (usually) at every 100-foot stake, the cross section being made perpendicular to the backbone line of the survey at that place, as is indicated by the dotted lines in Fig. 2. It is desired to plot on the map contours at each five-foot interval above the datum plane. Let Fig. 3 represent a typical cross section. Set the level (on its five-foot stick) at the stake S. The elevation of this stake given by the level party is (say) 169.4. The level therefore has an elevation of 174.4. If the level rod is moved up hill until it is found (by trial) that 4.4 mark is on a level with the telescope, then the base of the rod must have a level of 170 and must be on the 170-foot contour. Measure the distance *horizontally* from stake to rod and record as shown in Fig. 4. Leaving the level rod at that point, carry the stick and level up the hill until a level line strikes the top of the rod. The base of the stick is evidently on the 175-foot contour. Measure and record the distance as before. Carry the level rod to that point and in a similar manner determine the 180-foot contour if desired. The 165-foot contour is evidently 9.4 feet below the telescope when on the 5-foot stick at the center stake. The distance from the center to the 165-foot contour can thus be found. Lower contours can be similarly obtained. The results



F.W.SYKES.

AN OBSTRUCTION TO RAILROAD EXTENSION IN SOUTH AFRICA WHICH LONG PROVED INSUPERABLE

Gorge of the Zambesi River below the Victoria Falls, now spanned by a modern steel arch bridge. The waters of the river, two miles wide above the Falls, are here confined between 400-foot basalt walls not over 50 yards apart [illegible] of the Victoria Falls bridge, see Vol. VI, page 26.

should be plotted in a note-book ruled in quarter-inch squares, each side of a square representing 25 feet. The work will then be plotted on the scale of 100 feet per inch. If the successive stations are plotted *up* the page, the drawing will correspond with the points when looking ahead along the line. After plotting each section, the corresponding contours should be connected to form a sketch like Fig. 4. The crossing of the main line by a contour may be similarly determined. Fig. 4 is simply an enlarged detail of a sketch like Fig. 2. Although the Locke level is incapable of accurate leveling work, any error that may be made by the above method is confined to the station where it occurs and is not carried on and made cumulative. With reasonable care such inaccuracies can be kept within desired limits, while the rapidity is far greater than a more accurate method.

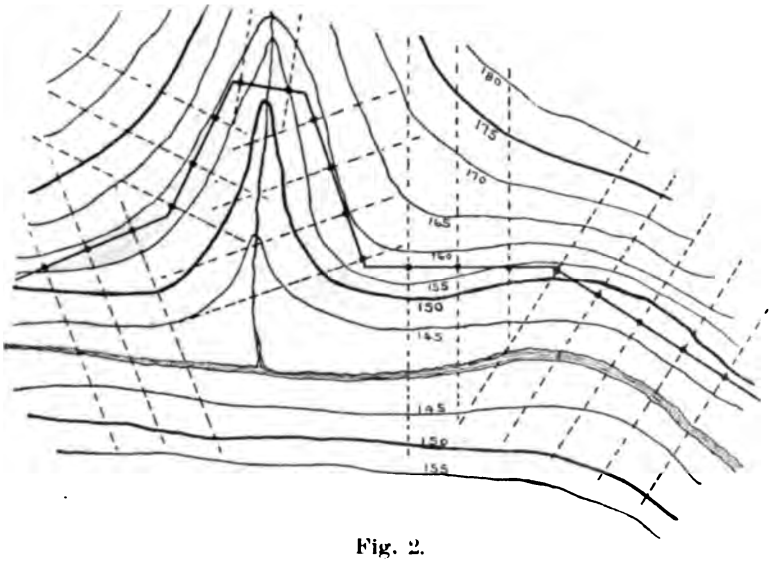


Fig. 2.

12. Stadia Method. This consists simply of a stadia survey of a long and narrow belt of country by the same general methods as those employed in ordinary stadia-topographical surveys. One advantage of this method is that the levels can be carried along very successfully as a part of the stadia work, if particular care is taken to always obtain practical agreement in the vertical angles for the foresight and backsight between consecutive stations. This will generally permit more rapid work, as the progress of the whole party is sometimes limited by the progress of the level party. The added cost of the level party is also saved. It is here assumed that the details of stadia work have already been studied and therefore no further discussion will be given of this very simple application of the general method. As in the previous method, the primary object of the survey is the preparation of a map showing the contours and required topographical features over the desired area.

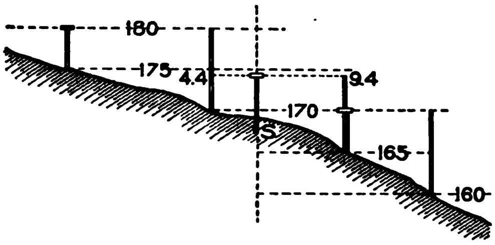


Fig. 3.

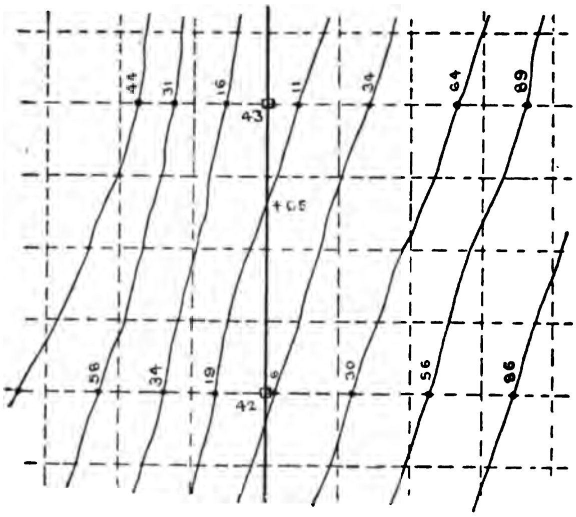


Fig. 4.

13. Party Required. It has been forcibly said that the only duty of the *chief-of-party* is to "keep his eyes open". The selection of the best route for a road so depends on a close study of the country that if the chief-of-party is required to do the work of transitman, as is sometimes the case, the work of either position is apt to suffer. The work of the transitman is so exacting that he should not be required to spend any time in studying out a route. Beside these two, there should be two flagmen, two chainmen, one stakeman and two or more axemen—depending on the wooded character of the country. On stadia surveys the flagmen and chainmen may be replaced by two or more rodmen; it is also economical to have a recorder, as it facilitates the progress of the whole party. The cross-section party should consist of a levelman, recorder, and two tapemen. This party *can* be cut down to three, or in an emergency two, but it is uneconomical in the long run. The level party will consist of a leveler and rodman. If the party is camping out, a cook and one or more teamsters will usually be required to handle the camp equipage, as it is unwise to require the surveyors to spend their time in such work.

14. Re-surveys. Much of the defective location of railroads is due to (1) deciding hastily on a general route, (2) then surveying a line through the belt with great detail and accuracy, (3) then locating the line substantially as first surveyed, because the line is fairly good (or at least not very bad), and also because of an unwillingness to throw away the detailed work of a large party for several weeks. Frequently a great amount of unnecessary and wholly useless detail is surveyed and plotted during the reconnoissance and preliminary surveys. These surveys should only include those salient facts which instantly stamp a route as being inferior or superior to another. Usually the general location of a large part of a route is self evident or may be determined after a brief examination. But there are generally places along the line where for a few miles a hasty examination of two or three lines is not only justifiable but is the only proper course. Two or more of these short loops may show advantages so evenly divided that a more elaborate survey is necessary to decide between them. Even after the location survey has been made, or even after construction has begun, changes are often proper, but if the preliminary surveying has been well done only minor changes should be needed. A few hundred dollars spent on extra surveying is a wise investment considering the great probability of an immediate saving of as many thousands in construction or of an operating advantage whose annual value might be as great as the cost of the extra surveying.

LOCATION SURVEYS.

15. Selecting a Route. Much of the railroad location of the country has been done by picking out the line on the ground, even making it follow in places the backbone line of the preliminary survey, running from one course to the next by means of suitable curves. In the hands of a good engineer the method is not necessarily very bad, but it is much improved by the following modification. *Paper location.* The work of the preliminary survey is carefully plotted from the transit notes and cross-section book to a scale of 200 feet per inch. On this map may be plotted one or more trial location lines. Each of these consists of circular curves joined by tangents. The location line must pass through any predetermined points and yet join them by lines which will give the best location, considering the conflicting interests as described in section 1. Within the limits of the preliminary map several locations are generally possible and one great element of the value of such a map lies in the ease with which several routes may be laid out and compared. Profiles may be drawn for each line laid down by noting the intersection of the line with each contour. Drawing on the profile the required grade line will give a relative idea of the amount of earthwork required. The method is especially valuable when "development" is necessary. Although such a line must sometimes be laid out by a bold and apparently unsystematic trial of a route, yet some approach to a systematic solution may be made as follows: Assume that the maximum ruling grade has been determined as 1.2 per cent, and that the contours have, as usual, a five-foot interval. It will require 417 feet of 1.2 per cent grade to rise five feet. Set a pair of dividers so that they will step off spaces of 417 feet on the map. Starting on a contour at the required beginning of a grade, swing the dividers so that they will just reach the next contour and continue to step off such spaces. Joining these points, such a line would be a purely surface line, would probably be very crooked and otherwise unsuitable, but it probably would be suggestive of a practicable route. After locating on the map the best obtainable line, it should then be transferred to the ground. Measure to scale the lengths of all "tangents" (the straight lines joining the curves), and the radii and lengths of all curves. Instead of scaling off the *length* of a curve, it may be more accurate to measure with a protractor, or with a scale of chords, the angle between the tangents at each end of the curve, and from the angle and the radius compute the length. Usually the located line will lie fairly close to the preliminary line—close enough so that tie lines may readily be run between them. These should be scaled from the map. To prevent the accumulation of error due to inaccuracies, the length (or radii) of curves or the length of tangents should be altered if necessary so as to make the location check on the ground with the positions of the stakes of the preliminary survey. The method of making such modifications will be taken up later.

16. Surveying Methods. Only the most precise work with a transit can be tolerated. The compass needle is only to be used as a check, but its use for this purpose should be insisted on, as it frequently detects a gross error. Transit stations should be marked by "hubs" and "witness stakes" (Section 10). Reference stakes should also be set at places as near as possible to the principal stations and yet outside of the line of all earthwork operations, so that at any stage of the construction the positions of the original stakes may be easily recovered. The link chain as a measurer has now been practically discarded for the steel tape. Fractions of a foot are measured in tenths and hundredths rather than in inches. The *personnel* of the party will be almost identical with that of the preliminary survey party except that the cross section party will be replaced by the slope-stake party, whose duties are similar, but who generally use a level on a tripod rather than a hand level. The description of the duties of the slope-stake party will be deferred to a later chapter. The leveling party should establish "bench-marks" at frequent intervals along the line. A spike driven in the roots of a large tree is one of the best and easiest established of marks in rural districts. A mark on any large masonry structure, such as a bridge abutment or a building, should be obtained when possible. Levels should be taken to hundredths of a foot on turning points and bench marks. Some engineers read to thousandths of a foot, but when it is considered that one division of a level bubble usually corresponds to 30″ of arc, and that at a distance of 150 feet a movement of 30″ of arc will correspond to .0218 foot on the rod, an error of level amounting to a very small fraction of a division will make an error of several thousandths or even a hundredth. Therefore unless unusual care is taken in handling the level, it is a useless refinement to read the rod to thousandths. In reading elevations of the surface of the ground, the nearest tenth of a foot is sufficiently accurate. The complete details of location surveys can only be appreciated after the subject of railroad curves has been studied, and they will not therefore be further elaborated here.

SIMPLE CURVES.

17. Method of Measurement. The alignment of a track is the geometrical form of the line midway between the two rails. Such a center line may be a straight line, a simple curve or a curve of double curvature, but it simplifies matters to consider always the *horizontal projection* of such lines. Their vertical projections are considered separately when it is necessary. Curves are sometimes designated by their radius or by the degrees and minutes subtended by a unit chord. Nearly all railroad curves have such long radii that it is impracticable to use the center. Therefore all work is done at the circumference in accordance with geometrical principles which will now be described. If AB, Fig. 5, is a chord of unit length, then D is called the *degree of curve* for the radius R.

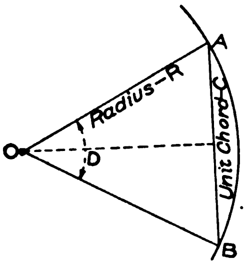


Fig. 5.

$$AO \sin \frac{1}{2} D = \frac{1}{2} AB = \frac{1}{2} C.$$

$$\therefore R = \frac{\frac{1}{2} C}{\sin \frac{1}{2} D} \qquad (1)$$

which becomes by inversion

$$\sin\frac{1}{2}D = \frac{C}{2R} \qquad (2)$$

The length of the unit sub-chord varies somewhat with custom. The almost invariable practice in the United States is to use a unit chord length of 100 feet. Substituting $C = 100$ in equation 1, and successively assuming values from 0° 01′ up to 12° 0′ varying by single minutes, and with larger intervals for higher degrees which are very seldom used, the radius of almost any curve may be tabulated for ready and convenient use. Such a table is found in Table I, which also gives the logarithm of each radius. A very common rule, which is approximate but accurate enough for many uses, is as follows, using the same notation as before:

$$R = \frac{5730}{D} \qquad (3)$$

18. Sub-Chords. It often becomes necessary to lay off a chord length which is less than 100 feet and to know the angle subtended at the center. Since a chord is shorter than its arc, it also follows that the sum of the four equal chords in Fig. 6 is also shorter than the total arc although they are evidently longer than the 100′ chord. But it is found more convenient to say that the chord has a *nominal* length (in this case) of 25 feet. As in equation (2) we may derive

$$\sin\frac{1}{2}d = \frac{c}{2R} \qquad (4)$$

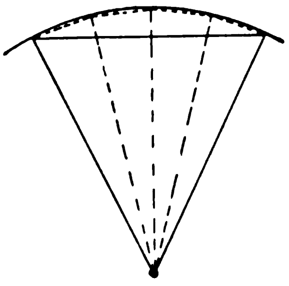


Fig. 6.

In which d is the angle subtending the sub-chord whose *true* length is c. By inversion we have

$$c = 2R\sin\frac{1}{2}d \qquad (5)$$

Calling the *nominal* length c', we have the proportion

$$c' : 100 :: d : D$$

EXAMPLES FOR PRACTICE.

1. What is the true length of a chord of a 3° 30′ curve whose nominal length is 40 feet? From the above proportion, $d = \frac{40}{100} D = 0.40 \times 3.5 = 1.4° = 1° 24′$. Substituting in equation 5, we have

$$c = 2 \times 1637.3 \times \sin \frac{1}{2} (1° 24′) = 40.005.$$

Note that the *excess* over 40 feet is very small—about one-sixteenth of an inch. It is always small for low degrees of curvature. In the following example it is far greater.

2. What is the true length of a chord of a 12° curve whose nominal length is 60 feet? *Ans.* 60.070. In this case it would be a gross error to neglect to allow for this difference.

19. Length of a Curve. The length of a curve is always considered to be the quotient of 100Δ ÷ D, in which Δ is the total central angle of the curve or the angle between the terminal tangents. The mean length of the two rails of a curve is always a little in excess of this, but the excess is always so small that it has no practical importance. It merely adds an insignificant amount to the length of rail required. *Example.* A 4° curve begins at Sta. 16 + 80 and runs to Sta. 21 + 35. The *nominal* length of the curve is 455 feet. The actual arc (which is the mean of the two rail lengths) is

$$4.55 \times 4° \times R \times \frac{\pi}{180°} = 455.09$$

which shows that the excess in this case = .09 foot, a little over an inch.

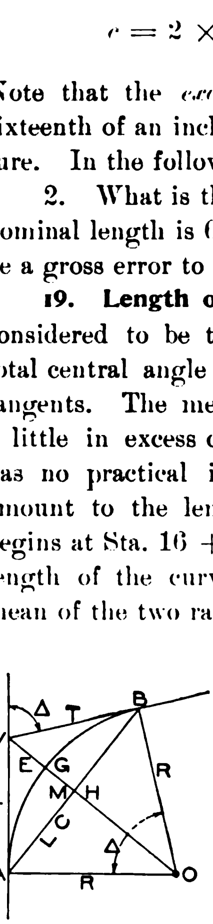


Fig. 7.

20. Elements of a Curve. The following fundamental relations apply to all curves. See Fig. 7. The beginning of the curve, A, is called the *point of curve*, PC. The other end of the curve at B is called the point of *tangency*, PT. The intersection of the two tangents is called the *vertex* (V). The *central angle*, Δ, is the angle at V between the tangents, and it is equal to the angle at the center, O, between the radii drawn to the PC and PT. The two equal tangents AV and BV are called *tangent distances*, T. The chord AB is called the *long chord*, LC. The distance HG from the middle of the long chord to the middle of the arc is called the middle ordinate, M. The distance GV from the middle of the arc to the vertex is called the *external distance*, E. From trigonometry the following commonly used relations are easily derived.

$$T = R \tan \frac{1}{2} \Delta \qquad (6)$$

$$LC = 2R \sin \frac{1}{2} \Delta \qquad (7)$$

$$M = R \operatorname{vers} \frac{1}{2} \Delta \qquad (8)$$

$$E = R \operatorname{exsec} \frac{1}{2} \Delta \qquad (9)$$

(*Note.* The *versed sine*, abbreviated to *vers*, and the *external secant*, abbreviated to *exsec*, are trigonometrical functions which are not commonly used except in railroad work, and some works on trigonometry omit their discussion. An inspection of the figure readily shows that $\operatorname{vers} a = 1 - \cos a$, and that $\operatorname{exsec} a = \sec a - 1$.)

From trigonometry we may derive the general equation that $\tan a \div \operatorname{exsec} a = \cot \frac{1}{2} a$. Therefore, by dividing equation 6 by equation 9 and transposing we obtain

$$T = E \cot \frac{1}{4} \Delta \qquad (10)$$

21. Elements of a 1° Curve. The various elements of a curve are exactly proportional to the radius and nearly proportional to the degree of curve. Therefore if the tangents, external distances and long chords are computed from equations 6, 7 and 9 for various values of Δ from 1 to 91 , varying by 10′, then an approximate value for any degree of curve and value of Δ may be found by taking out its value for a 1 curve (by interpolation if necessary) and then dividing that value by the degree of curve. For low degrees of curvature the inaccuracy of this method is usually small

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enough to be neglected. Even for sharper curvature the values obtained are accurate enough for approximate work. For absolutely accurate values equations 6 to 9 should be used, but the tabular values, found in table II, may always be used as a check.

22. Numerical Examples. 1. What is the tangent distance of a 3° 10′ curve whose central angle is 16° 26′?

Solution. log R = 3.25757

$\frac{1}{2}\Delta = 8^\circ\ 13'$, log tan = 9.15956

Tangent = 261.30 log 261.30 = 2.41714

Approximate solution. Interpolating in table II between the values for $\Delta = 16^\circ\ 20'$ and $16^\circ\ 30'$ we have the value 827.36 as the tangent distance for a 1° curve when the central angle is 16° 26′. Dividing 827.36 by 3.1666 (3° 10′) we have 261.27 as the approximate value. The inaccuracy is about one-hundredth of one per cent or in absolute value about three-eighths of an inch.

2. Compute the external distance and the long chord for the above curve, both accurately and approximately.

3. Two tangents make an angle of 18° 24′. It is desired to run a line which shall pass 21.2 feet from the vertex of the curve. What is the required radius and the resulting tangent distance? *Indicated solution.* The known quantities are E and Δ; from equation 10 we may derive T; then with T known and Δ a given quantity, we may compute R by an inversion of equation 6.

METHODS OF FIELD WORK.

23. Location of Points by Deflections. The angle between a tangent to a curve at any point and a secant from that point to any other point of the curve, is measured by one-half of the arc between those points. It is also equal to one-half of the angle between the radii to those points. On this fundamental geometrical proposition depends the whole science of circular-curve location. As a corollary, the angle between two secants intersecting on a point of the curve is measured by one-half of the intercepted arc or by one-half of the angle between the radii drawn to the ends of the intercepted arc. Applying these statements to Fig. 8 we have

$$\text{TO}a = \frac{1}{2}\text{OC}a$$

$$a\text{O}b = \frac{1}{2}a\text{C}b$$

$$b\text{O}d = \frac{1}{2}b\text{C}d$$

If $\text{O}a = 100$ feet, then by definition, the angle $\text{OC}a = \text{D}$, and the angle $\text{TO}a = \frac{1}{2}\text{D}$. Likewise if the chord $ab = 100$ feet, then the angle $a\text{C}b = \text{D}$ and the angle $a\text{O}b = \frac{1}{2}\text{D}$. bd is a subchord subtending the angle d, and the angle $b\text{O}d = \frac{1}{2}d$. Therefore if a transit is set up at the point O, any point of the curve may be determined by measuring the proper chord length from O in a direction determined by swinging an angle from the tangent OT equal to *one-half* of the angle measured at C between O and the desired point. But the measurement need not be made directly from O if other points have already been determined; b may be determined from a and d from b. Since it is generally impracticable to locate more than 500 feet of curve from any one point, on account of natural obstructions (and sometimes the distance is very short), the transit must be moved up to a new station already established on the curve. But the same principles will apply and may be repeated indefinitely.

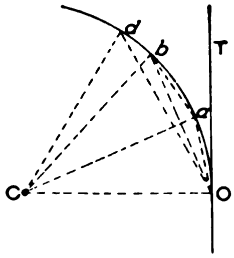

Fig. 8.

24. Computing the Deflections. If the point of curve is less than 100 feet from the last regular station, the remainder of the 100 feet must be laid off as a subchord. One-hundred-foot chords are set off until a station is reached which is within 100 feet of the end of the curve or (numerically) until the degrees of central angle remaining is less than D. That remainder is the angle for the final subchord. The foregoing may be illustrated by a numerical case: A 4° curve is to begin at Sta. 24 + 40. The central angle is 18° 40′. Compute the deflections. The first station point is 60 feet beyond the point of curve. The subchord angle is therefore $\frac{60}{100} \times 4° = 2.4° = 2° 24'$. The deflection from the tangent is one-half of this or 1° 12′. The deflection for the P.T. is one-half of the total central angle or 9° 20′. Subtracting 1° 12′ we have left 8° 08′, which will allow for four deflections of 2° each and 0° 08′ over, which will require a chord $= \frac{0° 08'}{2°} \times 100 = 6.67$ feet. The curve will therefore end at Sta. 29 + 6.67. This may be verified or otherwise computed as follows: $\frac{18° 40'}{4°} = 4.66667$, the total *nominal* length of the curve in station lengths of 100 feet. That is, the length will be 466.67. The first subchord is 60 feet; then four chords of 100 feet; then a final subchord of 6.67 feet. The deflections may be tabulated as follows:

P.C. Sta. 24 + 40 0°
25 0° + 1° 12′ = 1° 12′
26 1° 12′ + 2° = 3° 12′
27 3° 12′ + 2° = 5° 12′
28 5° 12′ + 2° = 7° 12′
29 7° 12′ + 2° = 9° 12′
29 + 6.67, 9° 12′ + 0° 08′ = 9° 20′, which is one-half of 18° 40′ as it should be.

25. Instrumental Work. The above numerical case is comparatively simple. When the degree of curve is an odd quantity and when difficulties of location require that the transit be set up at substations on the curve, then the numerical work, although worked out on precisely the same principle, is much greater and chances for numerical error are greater. The following rule for instrumental work is as simple as any for the simple cases and is far better for the more complicated cases. Compute the deflections for all stations and substations as illustrated above. Set up the transit at the P.C., and locate from it all stations that may be conveniently reached. Then move up the transit to a forward station and use the following rule:

When the transit is set at any forward station, backsight to ANY *previous station with the plates set at the deflection angle* *for the station sighted at. Plunge the telescope and sight at any forward station with the deflection angle computed for that station.*

The student should verify for himself the truth of this rule by drawing out a simple case and noting the angles both for foresight and backsight for any station, when the transit is located at any station.

Curve location requires extreme care on the part of field men, for a very slight inaccuracy is apt to be multiplied until the error is intolerable. The transit should be very carefully centered over hubs, which should be referred to points which will not be disturbed during construction.

26. Special Methods of Location. The above method, using a transit and tape, is the ordinary and preferable method, but it is sometimes necessary to lay out a curve when a transit is not at hand and there are sometimes special conditions when a modification of the above method will be more accurate. The engineer must have learned the fundamental principles of curve location so thoroughly that he may decide on the best method to use and even to invent some modification which may best suit the special case in hand. A few of these special cases will be described.

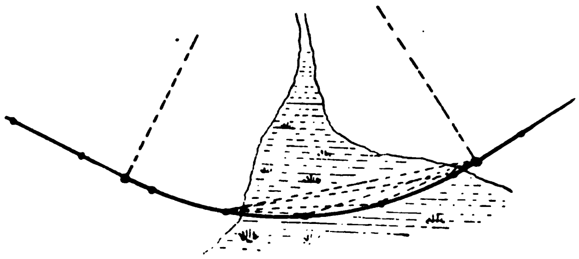

Fig. 9.

(*a*) *Using two transits.* The location may run over swampy ground where accurate chaining is impracticable. Some point of the curve beyond the swamp may be located, perhaps by triangulation, by computing its angle of deflection and the length of the long chord (equation 7). The point beyond the swamp may or may not be the P. T. Then set up two transits simultaneously at the stations located on firm ground •The deflection of each chord from the tangent to the curve at the instrument point, or from the long chord, is a simple matter of geometry (see § 23). A rodman can locate each point by placing himself at points where he is simultaneously in line for both transits.

(*b*) *By tangential offsets.* The solution of this as well as the following methods will be indicated by the lines in the figures. In each case the solution is an application of simple geometrical and trigonometrical principles. The solutions are somewhat lengthened, although not essentially modified, when the curve begins or ends with a subchord. In Fig. 10, for example

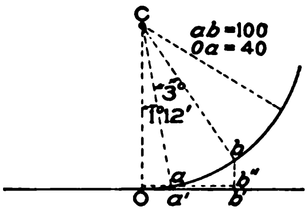

Fig. 10.

$$Ob' = Oa' + a'b' = 40 \cos 0^\circ\ 36' + 100 \cos (1^\circ\ 12' + 1^\circ\ 30')$$

$$bb' = b'\,b'' + b''b = 40 \sin 0^\circ\ 36' + 100 \sin (1^\circ\ 12' + 1^\circ\ 30')$$

and similarly for other points.

(*c*) *By middle ordinates.* Compute first the length of a long chord for *two* stations and the middle ordinate of such a chord. For subchords, compute the long chord and middle ordinates for an angle *twice* that subtended by the subchord. These distances should be laid off on the ground as indicated in the illustration. In Fig. 11, Oa'' is *half* the long chord for two stations and $a''a$ equals the middle ordinate for such a long chord. Lay off Oa on the tangent and measure out the offset $a''a$. Measure out aa' ($= a''a$) so that aa' is perpendicular to Oa', and produce Oa' to b. $Oa'' = Oa' = a'b$. Thus is b located, and c, d, etc., will be located similarly. In Fig. 12, an is half the long chord for twice the arc Oa, and On is its middle ordinate. Compute similarly zy and $z''z$, and lay off on the ground a and z. Compute, as in the regular case, aa' and za' ($= a'b$); b is then laid off as before.

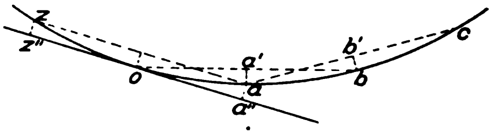

Fig. 11.

(*d*) *By offsets from the long chord.* The geometry involved is apparent from an inspection of Fig. 13, in which is shown the general case of a curve beginning and ending with a subchord. All of the above methods are mathematically perfect in theory, but when curves are thus laid out without the aid of a transit the work is apt to be inaccurate unless unusual care is taken.

27. Obstacles to Location. As in the previous section, the problems are usually simple examples in geometry and trigonometry, and the engineer must select the solution which will give the best result.

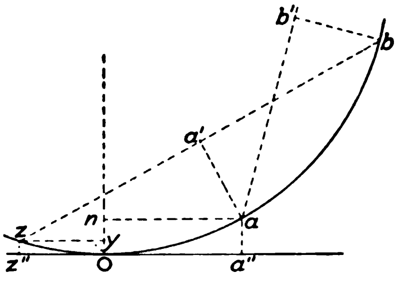

Fig. 12.

(*a*) *Vertex inaccessible.* The tangents are frequently fixed by certain conditions, and yet the intersection of the tangents is within a building or in some place where it is impossible to set up a transit. In the case shown in Fig. 14, the tangents are given by the points *a*, *b*, *n* and *m*. By measuring the angles *ba*V and *ab*V and the distance *ab*, the triangle *ab*V may be solved, and the distances *a*V and *b*V computed. The external angle at V is the sum of the angles at *a* and *b*, and equals the total central angle Δ. Having decided on the radius, the tangent distances are computed by equation 6, and then the differences B*b* and A*a* can be measured off and the P.C. and the P.T. are thus obtained. As a check on the whole work, the curve, run in by the usual methods, should end exactly at B, with the forward tangent coinciding in direction with B*n*.

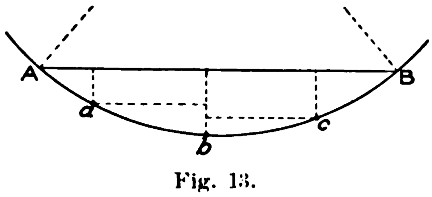

Fig. 13.

(*b*) *Point of curve, or point of tangent, inaccessible.* By making a diagram of the desired line with its obstructions, as in Fig. 15, the known and unknown quantities are readily determined, also their geometrical relations. For example, in the illustration the position of V (on the ground) is known, as is also the distance

AV. Then the *computed* position of A is known. Assume some angle a such that

$$R \text{ vers } a = As = nv = py$$

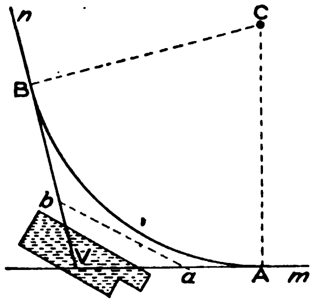

Fig. 14.

where s is in an accessible position. Then

$$ns = sp = R \sin a$$

and n and p can be located on the ground. Then, setting up a transit at n, and turning from the line np an angle of a, the tangent is determined and the remainder of the curve can be run in as usual. If the P.T. is inaccessible, the curve may be run in to some point m, from which, by similar calculations and field work, the point x is obtained, from which the tangent can be continued.

(*c*) *Middle part of curve obstructed.* The curve may be run as usual to some point n (Fig. 16) which should preferably, although not necessarily, be an even station. At n a chord nm may be run which will clear the obstruction. The angle between nm and the tangent is one-half the angle measured by the arc nm. From equation 7, the length of nm may be computed and then measured off, thus establishing the point m, from which the remainder of the curve is easily run in. As an illustration of the elasticity of this general method, it might under some conditions be easier to run the dotted curve having the same radius as the required curve could then be found by using the same geometrical principles used in §26 *d*.

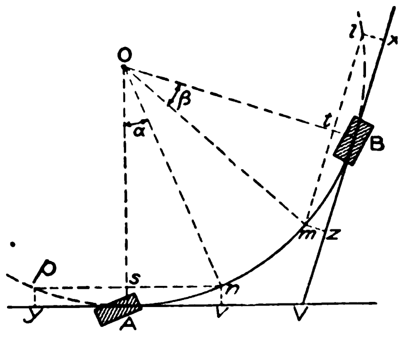

Fig. 15.

28. Numerical Examples. All problems have hitherto been so very simple that nothing has been said about the details of solv-

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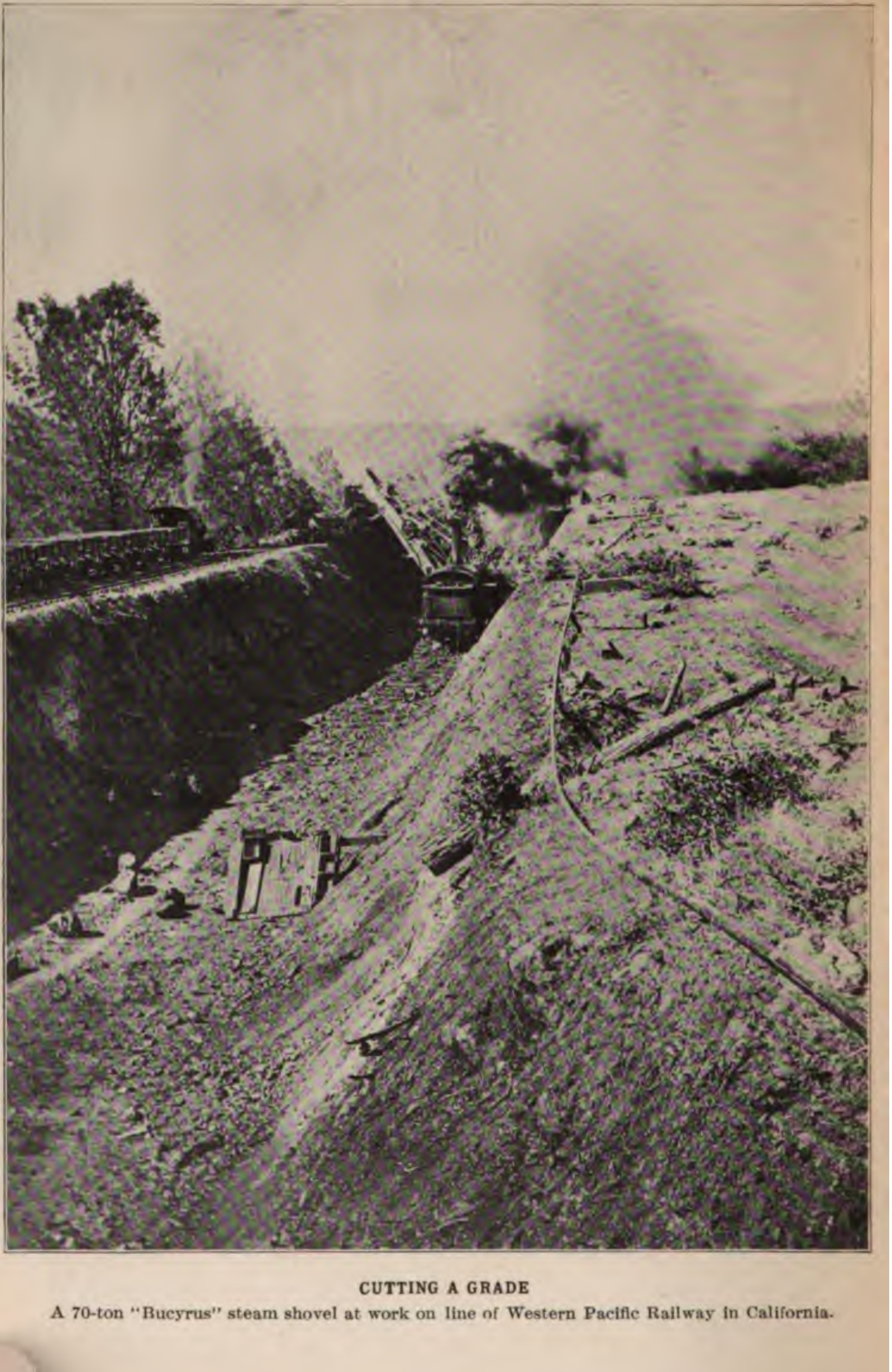


CUTTING A GRADE

A 70-ton "Bucyrus" steam shovel at work on line of Western Pacific Railway in California.

ing numerical problems. But as problems become more complicated, the greater becomes the value of a systematic method of solution, which may be readily reviewed, checked and studied for the discovery of a possible error. Logarithms should almost invariably be used for multiplication and division, for they are great time-savers. Even if the student is unaccustomed to them, it pays to become familiar with them. Such methods will be used in the following solutions and the student is urged to solve all such problems similarly.

1. In a case similar to that sketched in Fig. 14, ab was measured as 476.25; the angle Vab was measured as 24° 18′, and the angle Vba 34° 22′. The curve is to be a 3° 30′ curve. Its radius is therefore 1637.3. $\Delta = 24° 18' + 34° 22' = 58° 40'$. Compute aA and bB.

		Logarithms.
Equation 6.	R (3° 30′)	3.21412
	$\tan \frac{1}{2}\Delta = \tan 29° 20'$	9.74969
	$T = 920.04$	2.96381
$aV = ab \dfrac{\sin 34° 22'}{\sin 58° 40'}$	$ab = 476.25$	2.67783
	log sin 34° 22′	9.75165
	co-log sin 58° 40′	0.06836
	$aV = 314.74$	2.49795
	Tan $AV = 920.04$	
	$aA = 605.30$	
$bV = ab \dfrac{\sin 24° 18'}{\sin 58° 40'}$	$ab = 476.25$	2.67783
	log sin 24° 18′	9.61438
	co-log sin 58° 40′	0.06846
	$bV = 229.45$	2.36068
	Tan $BV = 920.04$	
	$bB = 690.59$	

2. Example as in Fig. 15. D = 3° 20′. Δ = 23° 40′. It is estimated that at v, 180 feet back from V, the line np will probably clear the obstruction at A; ns is the difference between 180 and the computed tangent distance AV; $ns = R \sin a$. Then $nv = py = R \text{ vers } a$. Locate n by the offset vn, and make a similar offset at y. If this line does not clear the obstruction, another value of a (probably greater) should be assumed and new values for Av and vn computed. Compute the numerical values as above.

29. Modifications of Location. Only a few of the very many changes which are at times required will here be given. They are all solvable by a few principles of geometry and trigonometry. The occasion for many such changes is the adjustment of the inaccuracies of a "paper location."

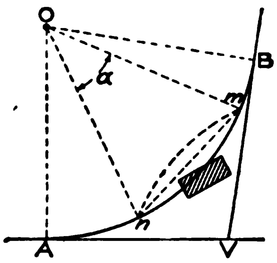


Fig. 16.

(*1*) *To move the forward tangent parallel to itself a distance x, the radius remaining unchanged.* See Fig. 17. Every point of the curve is moved parallel to the first tangent a distance AA' = BB' = VV' = OO'.

$$AA' = \frac{B'n}{\sin nBB'} = \frac{x}{\sin \Delta} \qquad (11)$$

(*2*) *To move the forward tangent parallel to itself, the point of curvature remaining unchanged.* Since the central angle (Δ) is unchanged, the curve and all its parts are simply enlarged or reduced according to some ratio, as is apparent from Fig. 18. The known quantities are the change in the tangent x' (or x''), the central angle Δ and the original radius R.

$$VV' = \frac{V'h}{\sin hVV'} = \frac{x'}{\sin \Delta} \qquad (12)$$

Then the new tangent distance AV' = AV + VV'. The triangle BmB', being similar to the triangle AO' B', is isosceles and Bm = B'm. Then the new radius

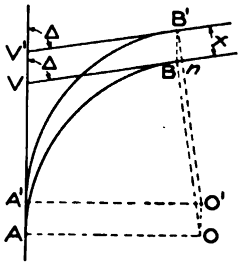


Fig. 17.

$$R' = R + mB = R + \frac{B'r}{\text{vers } B'mB} = R + \frac{x'}{\text{vers}\,\Delta} \qquad (13)$$

The modifications of this solution, when the tangent is moved toward the center, are very simple and are apparent from the figure.

(3) *To change the direction of the forward tangent at the point of tangency.* The central angle scaled off from the paper location might have an error which would be best corrected by this means. This solution is but one of a large class in which the central angle is modified. The required change (a) in the central angle is one of the given quantities. R, Δ, AV and BV are also known. In Fig. 19, $\Delta' = \Delta - a$; $Bx = R \text{ vers } \Delta$; $Bx = R' \text{ vers } \Delta'$

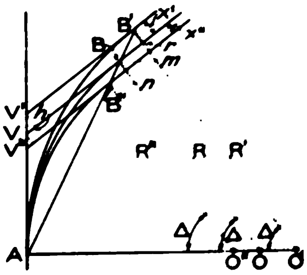


Fig. 18.

$$\therefore \quad R' = R \frac{\text{vers } \Delta}{\text{vers } (\Delta - a)} \qquad (14)$$

Also, since $Ax = R \sin \Delta$ and $A'x = R' \sin \Delta'$, we have

$$AA' = A'x - Ax = R' \sin \Delta' - R \sin \Delta \qquad (15)$$

30. Examples. 1. Given a 4° 20′ curve with a central angle of 18° 28′. It is required to move the forward tangent parallel to itself 12 feet. How much is the change of the P.C. (the distance AA′ in Fig. 17)?

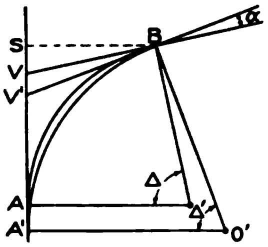


Fig. 19.

2. Given the same curve as above, it is required to move the tangent *toward* the center 12 feet, but without changing the P.C. What will be the changes in the tangent distance and the radius?

3. Given the same curve as above, it is required to *diminish* the central angle by 0° 22′, but retaining the same P.T. What will be the new radius and the change in the P.C.?

COMPOUND CURVES.

31. Definition. Compound curves consist of a succession of two or more curves of different radii which have a common tangent where they meet. They *may* be laid out by the same method as simple curves, but there are certain geometrical relations existing between the parts of a compound curve which greatly facilitate the computations, especially when any modifications are required. In the following demonstrations R_1 and R_2 will always represent the smaller and larger radii respectively, no matter which succeeds the other. Δ_1 and Δ_2 will always represent the corresponding central angles. Although R_2 is always larger than R_1, Δ_2 may or may not be larger than Δ_1. T_2 is always adjacent to the larger radius R_2 and is always larger than T_1.

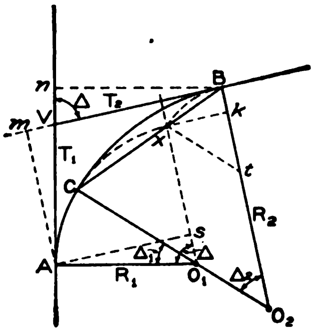


Fig. 20.

32. Mutual Relations of the Parts of a Compound Curve of Two Branches. The curve is illustrated in Fig. 20, in which AC and CB are the two curves with radii of R_1 and R_2 respectively. Therefore by the above definitions the other functions are as indicated in the figure. Produce the arc AC until the angle $CO_1 x = \Delta_2$. Then, by similar triangles, the chord Cx produced must intersect B. Also, if xt is drawn parallel to CO_2, it will equal Bt and the angle xtB will equal Δ_2. Then draw As and xk perpendicular to $O_1 x$. Then

$$\begin{aligned} Bk &= xt \text{ vers } xtB &&= (R_2 - R_1) \text{ vers } \Delta_2 \\ xs &= AO_1 \text{ vers } AO_1x &&= R_1 \text{ vers } \Delta \\ Am &= AV \sin AVm &&= T_1 \sin \Delta \\ Am &= Bk + xs \end{aligned}$$

$$\therefore T_1 \sin \Delta = R_1 \text{ vers } \Delta + (R_2 - R_1) \text{ vers } \Delta_2 \quad (16)$$

By drawing a few additional lines in the figure, it may similarly be proved that

$$T_2 \sin \Delta = R_2 \text{ vers } \Delta - (R_2 - R_1) \text{ vers } \Delta_1 \quad (17)$$

By algebraic transformation we may derive from equations 16 and 17 the following useful relations. The details of the derivation of these equations is suggested as a profitable exercise for the student.

$$R_2 = R_1 + \frac{T_1 \sin \Delta - R_1 \text{ vers } \Delta}{\text{vers } (\Delta - \Delta_1)} \quad (18)$$

$$R_2 = \frac{T_1 \sin \Delta \text{ vers } \Delta_1 - T_2 \sin \Delta \,(\text{vers } \Delta - \text{vers } \Delta_2)}{\text{vers } \Delta_2 \text{ vers } \Delta_1 - (\text{vers } \Delta - \text{vers } \Delta_1)(\text{vers } \Delta - \text{vers } \Delta_2)} \qquad (19)$$

33. Modifications of Location. As in § 29, only a few of the most common modifications will be here illustrated.

1. *To move the forward tangent parallel to itself, but without changing the radii.* From Fig. 21 we derive

$x = O_2s - O_2's' = (R_2 - R_1) \cos \Delta_2 - (R_2 - R_1) \cos \Delta_2'$, from which

$$\cos \Delta_2' = \cos \Delta_2 - \frac{x}{R_2 - R_1} \qquad (20)$$

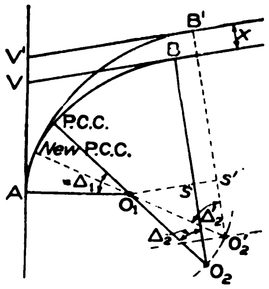


Fig. 21.

Fig. 21 shows the tangent as having been moved *outward*; in such a case the P.C.C. (which means the "point of compound curvature") is moved *backward* along the sharper curve. If it is desired to move the tangent toward the center, the required equation may be found by transposing Δ_2 and Δ_2' in equation 20. But in this case the sharper curve must be extended and the P.C.C. must be moved *forward*.

In case the larger radius comes first, the figure is apparently quite different, although a little study will show that the same principles apply. From Fig. 22 we derive

$x = O_1's' - O_1s = (R_2 - R_1) \cos \Delta_2' - (R_2 - R_1) \cos \Delta_1$ from which we have

$$\cos \Delta_1' = \cos \Delta_1 + \frac{x}{R_2 - R_1} \qquad (21)$$

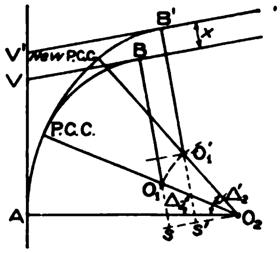


Fig. 22.

Fig. 22 shows the tangent as having been moved *outward*; in such a case the P.C.C. is moved *forward* along the easier curve which is extended. As before, if the tangent is moved *inward*, transpose Δ_1 and Δ_1' in equation. The P.C.C. will then be moved *backward* along the first curve.

(2) *To change the radius of one of the curves without changing either tangent.* The requirements will be apparent from a "paper" solution. In Fig. 23 assume that the longer

radius, which comes first, is to be shortened by an amount equal to sO_2. The new center O′ must lie somewhere on the arc whose center is O_1 and whose radius is O_1s. It also must lie on a line which is parallel to AV and distant from it by R_2' which is equal to s + P.C.C. With O_2' as center, draw an arc from O_1 to m'. With O_2 as center, draw an arc from O_1 to m. It may be proved that mm' is parallel to AV. Draw the line O_1n perpendicular to AO_2.

$$mn = (R_2 - R_1) \text{ vers } \Delta_2;\ m'n' = (R_2' - R_1) \text{ vers } \Delta_2';\ mn = m'n'.$$

$$\therefore \text{vers } \Delta_2' = \frac{(R_2 - R_1)}{(R_2' - R_1)} \text{vers } \Delta_2 \qquad (22)$$

$$AA' = O_1n - O_1n' = (R_2 - R_1) \sin \Delta_2 - (R_2' - R_1) \sin \Delta_2' \quad (23)$$

34. Examples. 1. A 5° 30′ curve with a central angle of 16° 22′ has a tangent distance of 1800 feet from the P.C. to the vertex. At the P.C.C. the curve compounds into an easier curve. The total central angle is 30° 18′. What is the radius of the easier curve, and what is tangent distance?

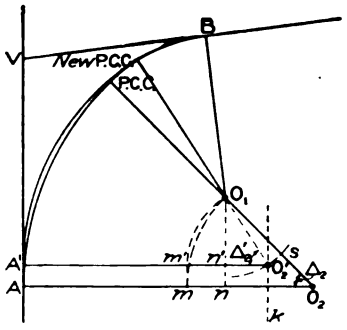


Fig. 23.

Answer. The given quantities are R_1 the *shorter* radius, Δ_1, Δ, and T_1; the required quantities are R_2 and T_2. By substituting the known quantities in equation 18 and then the computed valve of R_2 in equation 17, the required quantities are found. The student should perform this numerical work.

2. A 2° 30′ curve 450 feet long runs into a 5° 30′ curve 260 feet long. It is required to move the forward tangent *in* 6.4 feet, but without changing either radius. Required the change in the P.C.C. *Comment.* The solution evidently requires the indicated modification of equation 21. It should be noted that a practical solution always requires that the resulting value of the cosine shall be less than unity, which means that x can *never* be greater than $(R_2 - R_1)$, and also means that the sum of the cosines of the

original angle and its modified angle shall be less than unity. The linear change in the P.C.C. may be found by the formula

$$\text{Linear change} = (\text{angular change in degrees}) \times (\text{radius of curve}) \times \frac{\pi}{180}$$

3. Assume the same curve as in example 2, but that it is required to change the 2° 30′ curve to a 2° curve without changing the tangents. *Comment.* Fig. 23 may be made to apply by transposing the new and old larger radii, and their corresponding central angles. Note that when such changes are made in equation 22, the equation is unchanged. The effect on equation 23 is merely to change the algebraic sign, which means that the P.C.C. is moved *backward* instead of forward.

4. Draw a figure corresponding to Fig. 23 but showing a change in the smaller radius R_1.

TRANSITION CURVES.

35. Reasons for Use. When a train, or any other mass, is in motion it requires a definite force to make it move in a curved path. If the rails at opposite points of a railroad curve were at the same level, this centripetal force could only be furnished by the pressure of the wheel flanges against the outer rail. To avoid such a dangerous pressure, which would make high speed impracticable, the outer rail is made higher than the other. But it is manifestly impracticable to suddenly raise the outer rail at the beginning of a curve and lower it as suddenly at the end of the curve. There must be a "run-off" of considerable length. If this run-off were placed exclusively on the tangent, there would be an objectionable jar because the cars were tipped on a straight track where there is no compensating centrifugal force. If the run-off were entirely on the curve there would be a jar, because the centripetal force would not become fully developed at the beginning of the curve; and, therefore, a transition curve is used at the beginning and end of the curve. The transition curve is one whose rate of curvature gradually increases from nothing to the rate of the central part of the curve. If the super-elevation of the outer rail is begun at the beginning of the transition curve and is gradually increased as the curvature increases so that the proper super-elevation is attained at the end of the transition curve where the regular curve commences, then the theoretical requirements are satisfied. But there is another important reason for transition curves. On a curve the bogie trucks of a car make an angle with the axis of the car. If there were an instantaneous change from a straight track to the full degree of curvature the change of position of the truck would need to be accomplished in the time required to run the length of the wheel base of a truck. For a high-speed train this would be only a fraction of a second. On a transition curve this change of position is accomplished gradually and without jar. The amount of the required super-elevation will be discussed in the following sections.

36. Varieties of Transition Curves. A theoretical transition curve is very complicated and its mathematical treatment difficult. In any case it is located by points and the rails are *supposed* to conform to the theoretical curve between those points. Many railroads are using a system of multiform compound curves, beginning with an easy curve and at regular intervals compounding into a curve of sharper curvature, the increase in curvature being regular and uniform until the curvature of the regular curve is attained. This curve is also located by points; the points coincide so nearly with those of the theoretical curve that it would be difficult to detect the difference. When the rails are bent to those points they will not abruptly change their curvature at those points, but will assume a curve which will coincide very closely with the theoretical curve. The result is therefore a curve which is mathematically simple and yet a close approximation to the theoretical curve. These transition curves are also called "spirals."

37. Characteristics of these Spirals. In Table IV are given the deflections and other functions for three sets of spirals. The second set, which will best fit the most ordinary cases, consists of spirals with a uniform chord length of 25 feet, the curvature of the first chord being a 1° curve, of the second chord a 2° curve, etc. At the end of the fifth chord, which subtends a 5° curve, the spiral could be run on to a 6° curve, the uniform increase in rate of curvature would be 1° per 25 feet and the spiral would be 125 feet long, which would be a good length for a run-off. For a high-speed road, the rate of change should be less. For example, the first spiral changes 0° 30' per 25-foot chord. Therefore, to run on to a 4° curve there should be seven chords, of which the last should be a 3° 30' curve and the length of the spiral 175 feet. When the curvature is very sharp, the first spiral and even the second would require too long a spiral. For example, it would require a spiral 475 feet long (of the No. 1 type) to run on to a 10° curve; a No. 2 spiral would be 225 feet long. But if the curvature increases 2° per 25 feet, then the 10° curve could be reached with a spiral 100 feet long. Roads with 10° curves are presumably operated at slow speed over such curves and then the shorter spirals will suffice.

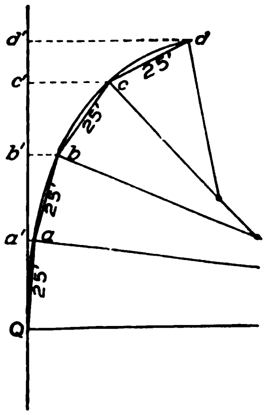


Fig. 24.

In Fig. 24, there is a very great distortion of the figure in order that the fundamental principles may be illustrated in a figure which is small enough to be placed on a page. The derivation of the deflections and the values of the total central angle, ϕ, and of the ordinates x and y to any point will not be here given. Those interested may find it given in Webb's Railroad Construction, §§ 44 to 48. The tables show for any point the total central angle, ϕ, between the original tangent and a tangent to the curve at that point; they also give the ordinate y from the point of tangency to the given point, measured along the tangent; also the ordinate x from the tangent out to the given point. The tables of deflections show successively for any point of the curve, the deflections from a tangent to the curve at that point to any other point of the spiral. The method of use of these spirals will follow.

38. Insertion of a Spiral between a Tangent and a Circular Curve. In Fig. 25 it has been necessary to make the distance MM' about 100 times its real proportional value in order to make the illustration distinct. The curve AMB is a simple circular curve joining the two tangents, such as would be used without spirals. If a suitable spiral is started at a suitable point Q, and run to some point Z, such that the total central angle of the spiral, ϕ, equals the angle ZO′N of the curve, and then the circular curve having a common tangent with the spiral at Z, is run to Z′, from which a similar spiral is run to the tangent at Q′, such a combination of curves will give the desired result. It is now necessary to compute the dimensions required to accomplish this result. The introduction of the spiral has the effect of throwing the curve away from the vertex by the amount MM′, which we will call m; it also reduces the central angle of the circular curve by 2ϕ. ZK is the x ordinate of the chosen spiral and QK the y ordinate.

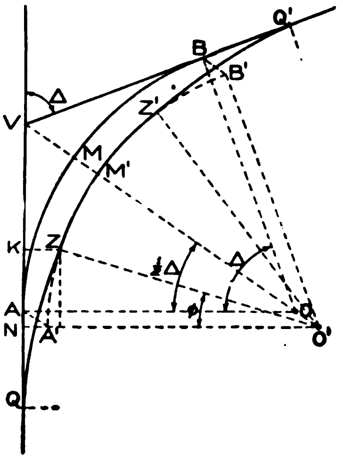


Fig. 25.

$$A'N = x - R \text{ vers } \phi.$$

Therefore,

$$m = MM' = AA' = BB' = \frac{A'N}{\cos \frac{1}{2}\Delta} = \frac{x - R \text{ vers } \phi}{\cos \frac{1}{2}\Delta} \quad (24)$$

$$NA = AA' \sin \frac{1}{2}\Delta = (x - R \text{ vers } \phi) \tan \frac{1}{2}\Delta$$

$$VQ = QK + KN + NA + AV$$

$$= y - R \sin \phi + (x - R \text{ vers } \phi) \tan \frac{1}{2}\Delta + R \tan \frac{1}{2}\Delta$$

$$= y - R \sin \phi + x \tan \frac{1}{2}\Delta + R \cos \phi \tan \frac{1}{2}\Delta \quad (25)$$

As a numerical solution of any problem will usually involve the determination of the value of A′N, equation 25 may be reduced from four terms to three as follows: Transform the equation above equation 25 to read

$$VQ = y + R\left(\tan \frac{1}{2}\Delta - \sin \phi\right) + A'N \tan \frac{1}{2}\Delta \quad (26)$$

$$VM' = VM + MM' = R \text{ exsec } \frac{1}{2}\Delta + \frac{x}{\cos \frac{1}{2}\Delta} - \frac{R \text{ vers } \phi}{\cos \frac{1}{2}\Delta} \quad (27)$$

$$AQ = VQ - AV = y - R \sin \phi + (x - R \text{ vers } \phi) \tan \frac{1}{2}\Delta \quad (28)$$

39. Example. It is required to join two tangents which make an angle of 28° 16′ by a 6° curve terminated by suitable spirals. A 1°-per-25-foot spiral with five chords will be suitable. Then, from Table IV, we find that

$$\phi = 3° 45', x = 2.999, y = 124.942 \text{ and } \frac{1}{2}\Delta = 14° 08'.$$

				Logarithms.
(Equation 24)			R	2.98017
			vers ϕ	7.33063
		2.045		0.31080
	x =	2.999		
	A′N =	0.954		9.97955
			$\cos\frac{1}{2}\Delta$	9.98665
(Equation 27)	m = MM′ = AA′ =	0.984		9.99290
			R	2.98017
			$\text{exsec}\,\frac{1}{2}\Delta$	8.49436
	VM =	29.821		1.47453
	m =	0.984		
(Equation 26)	V′M =	30.805		
y = 124.942	nat. tan. $\frac{1}{2}\Delta$ =	0.2518		
	nat. sin ϕ	0.0654		
		0.1864		9.27044
			R	2.98017
		178.080		2.25061
	(see above)		A′N	9.97955
			$\tan\frac{1}{2}\Delta$	9.40106
		0.240	AN	9.38061
	VQ =	303.262	R	2.98017
(Equation 28)			$\tan\frac{1}{2}\Delta$	9.40106
		240.564	AV	2.38123
	AQ =	62.698		

40. Insertion of Spirals in Old Track. An engineer frequently has occasion to insert spirals in track which was not so laid out. The simplest method from a mathematical standpoint is that given in the two previous sections. But this would involve moving the whole track for the entire length of the curve, and also, since it is apparent from Fig. 25 that the new line QZZ′Q′ is shorter than the old line QAMBQ, the method will involve rail cutting and the boring of holes for track bolts. The following method makes a slight sharpening of curvature of the middle circular section, which at the center is slightly *outside* of the old track, and so crosses the old line that the lateral deviation from the old line is always very small and the length of the new track need not differ appreciably from that of the old. The method of solution is indicated in Fig. 26.

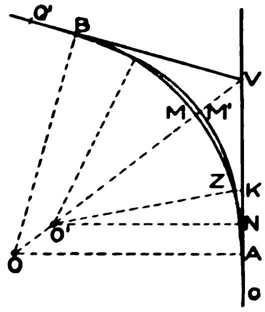


Fig. 26.

$$O'N = R'\cos\phi + x$$

$$O'V = O'N\sec\frac{1}{2}\Delta$$

$$= R'\cos\phi\sec\frac{1}{2}\Delta + x\sec\frac{1}{2}\Delta$$

$$m = MM' = MV - M'V$$

$$= R\ \text{exsec}\frac{1}{2}\Delta - R'\cos\phi\sec\frac{1}{2}\Delta - x\sec\frac{1}{2}\Delta + R \quad (29)$$

$$AQ = QK - KN + NV - VA$$

$$= y - R'\sin\phi + (R'\cos\phi + x)\tan\frac{1}{2}\Delta - R\tan\frac{1}{2}\Delta$$

$$= y - R'\sin\phi + R'\cos\phi\tan\frac{1}{2}\Delta - (R - x)\tan\frac{1}{2}\Delta \quad (30)$$

The length of the old line from Q to Q′ $= 2AQ + 100\frac{\Delta}{D}$.

The length of the new line from Q to Q′ $= 2L + 100\frac{\Delta - 2\phi}{D'}$. (31)

in which L is the length of each spiral.

41. Example. Assume that a track has been laid with a 6° curve for 39° 50′: It is desired to insert suitable spirals without altering the length of the old track. *Solution.* Unfortunately there is no method of solving this problem so as to obtain directly the revised value for the radius. The new radius will always be from 1 to 5 per cent shorter than the old. The larger the central angle, the less will be the difference. The only method is therefore to assume a value for R′, solve equation 30, and then compare the lengths of the new and old lines. If the difference is so small that it may be neglected, the problem is solved. If not, a slight modification of the new radius, such as experience in these computations will suggest, will usually give on a second trial a value which is sufficiently close. As a first trial for the above numerical case, we will assume a 6° 10′ curve for the new curve, with five chords of a 1°-per-25-foot spiral at each end.

				Logarithms.
(Equation 30) y =	124.942		R′ (6° 10′)	2.96828
			sin ϕ	8.81560
		60.797		1.78388
			R′	2.96828
			cos ϕ	9.99907
			tan $\frac{1}{2}\Delta$	9.55909
	336.080			2.52644
			R = 955.37	
			x = 3.00	
			952.37	2.97880
			tan $\frac{1}{2}\Delta$	9.55909
		345.064		2.53790
	461.022	405.861		
	405.861			
AQ =	55.161			

The length of the old curve from Q to Q′ is

$$100\frac{\Delta}{D} = 100\frac{39.83333}{6} = \qquad 663.889$$

$$2\ \text{AQ} = 2 \times 55.161 = \qquad 110.322$$

$$774.211$$

The length of the new curve from Q to Q′ is

$$
\begin{array}{lrr}
100\,\dfrac{\Delta - 2\phi}{D'} = 100\,\dfrac{39.8333 - 7.5^\circ}{6.167} = & 524.324 & \\
2L = 2 \times 125 = & 250.000 & \\
 & 774.324 & 774.324 \\
\text{Difference in length} = & & 0.113
\end{array}
$$

When it is considered that in that length of 774 feet there will be about 26 rail joints and that a stretching at each joint of about .004 foot will make up for this difference of length, it would be unnecessary to cut the rails.

To illustrate the method of adjustment if a more accurate value for R′ were required; note that in the above case the new curve is too long. If R′ is increased (say from D′ = 6° 10′ to D = 6° 9′), one term of equation 30 will be increased and one of them diminished, but the net change in the value of AQ is 0.748, which will increase the length of the old curve by 1.496. But such a change in D′ will increase the length of the new line by 1.419.

The revised length of the old line is, therefore,	775.707 and
the revised length of the new line is,	775.743
The revised difference is	0.036

The new line is still longer than the old, but the difference is insignificant (less than a fiftieth of an inch per joint). Of course D′ = 6° 9′ is the better value to use.

There is another method of introducing a spiral into old track without even changing the central part of the curve. The spiral runs into a curve which is sharper than the original curve which then compounds into the old curve. The solution of this method consists in so choosing and locating the spiral and the sharper curve that it will compound into the original curve. The details of this method will not be here given because, although it involves less track work at the start, it is a more complicated alignment to maintain and the method is inferior to the one previously given.

			Logarithms.
(Equation 29)		R (6)	2.98017
		exsec $\frac{1}{2}\Delta$	8.80356
60.776			1.78373
$R' =$ 929.57			
990.346		R'	2.96828
		cos ϕ	9.99907
		sec $\frac{1}{2}\Delta$	0.02678
	986.590		2.99413
		x – log =	0.47697
		sec $\frac{1}{2}\Delta$	0.02678
	3.190		0.50376
989.780	989.780		
$m =$ 0.566			

Note that the maximum lateral change in the track is less than seven inches.

On many railroads where there has been no pretense to using spirals the track foremen have produced nearly the same result in a rough uncertain way by throwing in the curve somewhat near the point of curve. This necessarily sharpens the curve further on, and thus substantially the same result is obtained as is described above but without any calculations or any theoretical accuracy. It is only by such means that a tolerable riding track can be produced when transition curves are not used.

42. Use of Transition Curves with Compound Curves. It is shown in the last few sections that the lateral deviation involved by the use of spirals is very small. Since compound curves are usually employed only when the location is difficult, it is best to make the calculations as if no spirals were to be used, except that approximate allowances may be made for such lateral changes as will be required. Then the changes can be computed as follows. Theoretically there should be a transition curve between the two branches of a compound curve, but when a train is already on a curve and the wheels are pressing against the outer rail, it will cause but little jar to merely increase the curvature. The introduction of such spirals will not be here given. Transition curves need not be used in running on to curves which are easier than 3° and even 4°. Therefore if one branch of a compound curve is of easy curvature, as is frequently the case, it will not be necessary to use a spiral at that end of the curve. Therefore two cases will be developed—using spirals at one end only, and at both ends.

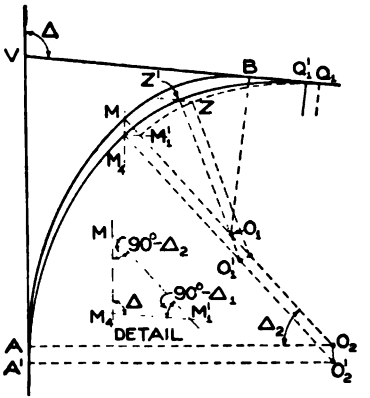


Fig. 27.

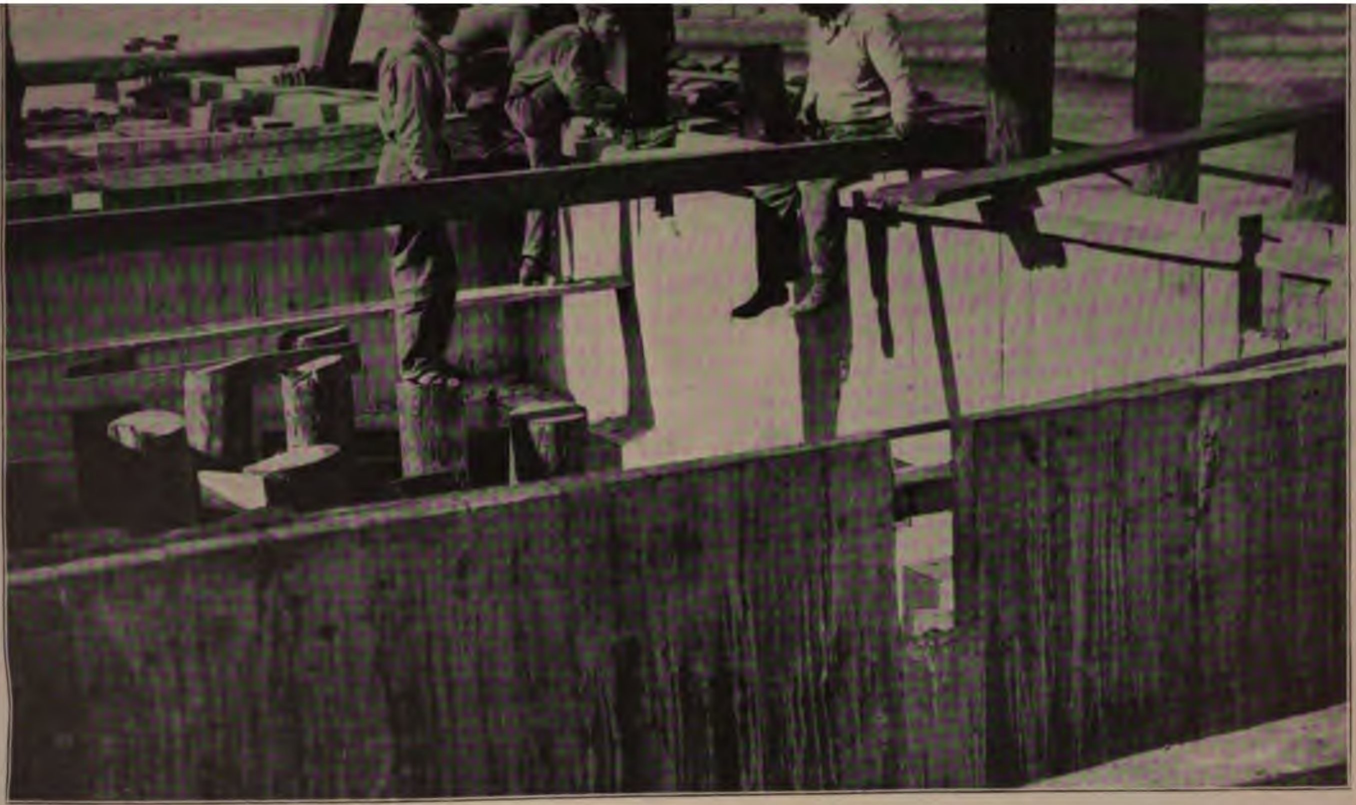
(*a*) *Spiral at one end only.* The method of §38 may be adopted by substituting Δ_1 for $\frac{1}{2}\Delta$ in equations 24 to 28. This would move the P.C.C. from M to M'. But since the two curves *must* be made to coincide, the sharper curve will be moved parallel to the tangent BV a distance of M'M, while the unchanged circular curve will be moved parallel to the tangent AV a distance MM_4. Calling $MM' = m_1$, we have

$$M_1'M_4 = MM_1'\frac{\sin M_1'MM_4}{\sin M_1'M_4M} = m_1\frac{\sin(90° - \Delta_2)}{\sin\Delta} = m_1\frac{\cos\Delta_2}{\sin\Delta} \quad \textbf{(32)}$$

$$MM_4 = m_1\frac{\cos\Delta_1}{\sin\Delta} \quad \textbf{(33)}$$

It should be noted that the new P.C. is at A' and that AA' = MM^4, while the P.T. is changed from B to Q_1', which equals $BQ_1 - QQ'$. BQ_1 is found from equation 28 and $Q_1Q_1 = M_1'M_4$ is found from equation 32.

(*b*) *Spiral at both ends.* Applying the method of §38 to each branch of the curve in turn by successively substituting Δ_1 and Δ_2 for $\frac{1}{2}\Delta$, we will obtain values for m_1 and m_2 which will in general differ considerably. But as before we may move each revised curve as shown in Fig. 28 and as computed in equations 34 and 35. Calling $MM_1' = m_1$ and $MM_2' = m_2$, and noting that the angle at M_1' (see the detail) $= 90 - \Delta_1$, the angle at M_2'



INSIDE OF COFFERDAM, SHOWING PIER PILES
Florida East Coast Railway extension to Key West.

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$= 90° - \Delta_2$ and the angle at $M_3 = \Delta$, we have the following:

$$M_1'M_3 = M_1'M_2' \frac{\sin(90° - \Delta_2)}{\sin \Delta} = (m_1 - m_2) \frac{\cos \Delta_2}{\sin \Delta} \quad (34)$$

$$M_2'M_3 = M_1'M_2' \frac{\sin(90° - \Delta_1)}{\sin \Delta} = (m_1 - m_2) \frac{\cos \Delta_1}{\sin \Delta} \quad (35)$$

As in the previous case, the position of the new point of the spiral is found in each case from one of the above quantities and the change computed from equation 28. Note that in one case the point of spiral is moved nearer to, and in the other case further away from, the vertex.

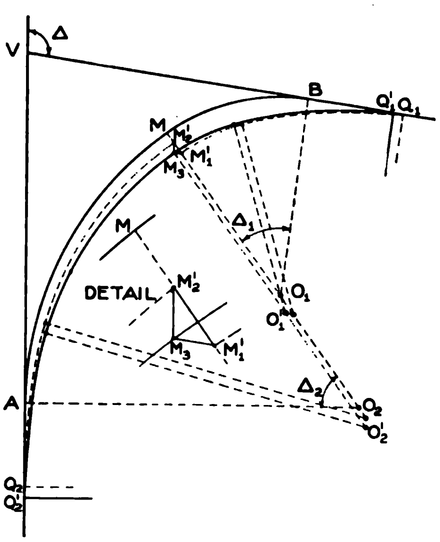


Fig. 28.

43. Example. Given a 7° 20′ curve for 29° 40′ followed by a 4° 10′ curve for 25° 20′. Required to introduce suitable spirals. Use spirals varying 1°-per-25-feet, with six chords for the 7° 20′ curve and three chords for the 4° 10′ curve. Using equation 24 successively with the two sets of figures we obtain $m_1 = 1.933$ and $m_2 = 0.323$. Then $(m_1 - m_2) = 1.610$. Substituting this value in equations 34 and 35 we obtain $M_1 M_3 = 1.776$ and $M_2'M_3 = 1.708$. Using equation 28, calling $\frac{1}{2}\Delta = 25° 20'$, we compute $AQ_2 = 39.128$. But we must *add* to this an amount equal to $M_2'M_3 = 1.708$, which makes $AQ_2' = 40.836$. Similarly, calling $\frac{1}{2}\Delta = 29° 40'$ in equation 28, we may compute $BQ_1 = 79.282$. But from this must be *subtracted* $M_1'M_3 = 1.776$, which makes $BQ_1' = 77.506$. The actual lateral change from the original point M is equal to $MM_2' + M_2'M_3 \sin \Delta_2 = 0.323 + 0.731 = 1.054$. The student should verify in detail all these calculations.

44. Field Work. When spirals form part of the original location, it is a useless refinement to locate all the chord points before earth work has been done. It is then sufficient to locate the beginning and end of each spiral with perhaps one intermediate point in case the spiral is very long. During the re-survey which immediately precedes track work, when the roadbed is graded and flat, the intermediate points are readily inserted. Referring to Fig. 25, the point Q would first be located at a distance VQ (see equation 25) from the point V. Assume, as in Fig. 25, a simple curve, 6°, with $\Delta = 32°$, and at each end a five-chord spiral with 1°-per-25-foot change of curvature. During the first location of the road it would be sufficient to locate the end of the spiral at point 5. From Table IV we find that the "deflection from the tangent at the point occupied when the instrument is at" Q and is "sighting at" 5 is 1° 22′ 30″. The ordinate y (QK in Fig. 35) is 124.942 and the distance out from the tangent $KZ = x = 2.999$. The total central angle to this point $(\phi) = 3° 45'$. The central angle left between Z and $Z' = 32° - (2 \times 3° 45') = 24° 30'$. Set up the instrument at Z. The "deflection from the tangent at the point occupied when the instrument is at" Z (which is here 5) and is "sighting at" Q is 2° 22′ 30″. Setting off that deflection on the transit so that when the instrument is turned around to the tangent it shall read 0°, the remaining central angle of 24° 30′ can be laid off in the usual manner. Again setting up the instrument at Z′ (which is point 5 of that spiral) the point Q′ is located with the same deflection (2° 22′ 30″) as the back sight from Z to Q. The distance from Z′ to the tangent VQ′ is likewise the same as $ZK = 2.999$.

When the intermediate points are to be located, the transit is set up at Q and the points are located by chord lengths of 25 feet and with deflections to the various points as given in the column under Q. When the circular curve ZZ′ has been located and the transit set up at Z′ and oriented so that it will read 0° when sighting along the tangent to the curve at Z′, then (using the deflections in the column under 5) the deflection to the point 4, 25 feet away, is 0° 37′ 30″; to the next point 3 it is 1° 11′ 15″; to the next point

2 it is 1° 40′ 00″; to the next point 1 it is 2° 03′ 45″; to the point of tangency Q it is, as given before, 2° 22′ 30″. Then the transit should be set up at Q′ and back-sighted on Z′ with a reading of 1° 22′ 30″. If the vertex of the curve V had already been previously located and the field work is accurate, the sighting on V should be 0°. Also the reading on any other point of the spiral should be as in the column of deflections under Q.

It sometimes becomes necessary to set up the transit at some intermediate point of the spiral, as (in the above case) the point 3. With the instrument set up at 3, use the deflections under the column 3 in the table, orienting the transit by a sighting at any known point with the transit set at the given deflection for that point. Then, when the telescope is turned to 0°, the transit will be sighting along the tangent to the curve at the point 3 and the deflection to any other point, forward or backward, will be as given in that column.

It may be noted that the deflections are given to single seconds of arc, which is of course very much closer than an ordinary transit can be used. But the location of spirals demands the closest work attainable with the ordinary field transit; and even though the transit is only graduated to single minutes, a fraction of a minute can be estimated in setting off a deflection and therefore the precise angles are given so that the closest attainable value of the true angle may be set off.

VERTICAL CURVES.

45. Reasons for Use. Although the change of direction of motion due to a change of grade is never as great as that of an ordinary horizontal curve, yet it is as necessary in one case as in the other to join the two grade lines by a "vertical curve." As in the case of horizontal curves, there is nothing which absolutely determines the rate of curvature. When the grades intersect over a summit a comparatively short curve is permissible, but when passing through a sag the upward bend of the track acts as a literal obstruction and therefore a much longer curve is necessary. Some roads adopt some uniform distance, such as 200 feet each side of the vertex, as the length of all such curves, regardless of the change in the rate of grade. A more logical method is to make the length a function of the change of grade. A very common rule is to make the length 100 feet for each tenth of one per cent of change of grade. For example, a one per cent grade descending into a hollow is followed by a 1.2 per cent grade climbing out of it; what should be the length of the vertical curve at the sag? The *change* of grade is the numerical sum of the grades or 2.2 per cent; therefore, by the above rule the length should be 2,200 feet. Such a length is excessive, but such a change of grade is also somewhat unusual and hardly to be found where the speed was high. For lower speed such a long vertical curve is not essential.

46. Geometrical Form of the Curve. The method of laying out such a curve is illustrated in Fig. 29, in which the grades are exaggerated enormously. The curve begins and ends at equal distances on each side of the vertex B, as at A and C. In the figure join A and C; bisect AC at *e*, join B and *e* and bisect B*e* at *h*. It may be proved mathematically that a parabola may be passed through *h* and tangent to AB and BC at A and C, and also that at *any* point, as at *n*, the distance to the tangent (*sn*) is proportional to the *square* of the distance from A. Expressing this statement algebraically, we have

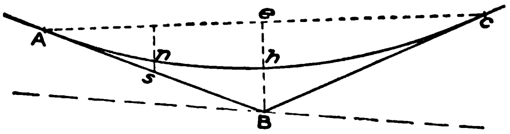


Fig. 29.

$$sn = eh\frac{(An)^2}{(Ae)^2} \qquad (36)$$

But since, in any given case eh is a constant and $(Ae)^2$ is a constant, we may say for any one curve that the correction from the straight grade line to the curve equals a *constant* times the square of the distance from one end of the curve.

47. Numerical Example. Assume that the intersection of the grades B comes at Sta. 15 + 40; that the grade AB is – 0.6 per cent and the grade of BC is + 0.8 per cent. Then if we adopt the rule of 100 feet for each tenth per cent of change, the curve must be 1,400 feet long, must begin at Sta. 8 + 40 and extend to Sta. 22 + 40. Assume that the elevation of B is

152.50; then the elevation of A is 152.50 + (7 × 0.6) = 156.7. Similarly the elevation of C is computed as 158.1. Then the elevation of *e* is the mean of these two, or 157.4. B*e* is therefore = 4.9 and *eh* = 2.45. Then *eh* ÷ $(\mathrm{A}e)^2$ = 2.45 ÷ 490000 = 000005, the constant which is to be multiplied by the square of the distance from A to obtain the correction from the straight grade up to the curve. The elevations of the several stations is most readily calculated in a tabular form such as is given below.

A. STATION
8 + 40, elev. = 152.50 + (7 × 0.6) = 156.70
9 = 156.70 − (0.6 × 0.6) + .000005 × 60^2 = 156.36
10 = 156.70 − (1.6 × 0.6) + .000005 × 160^2 = 155.87
11 = 156.70 − (2.6 × 0.6) + .000005 × 260^2 = 155.48
12 = 156.70 − (3.6 × 0.6) + .000005 × 360^2 = 155.19
13 = 156.70 − (4.6 × 0.6) + .000005 × 460^2 = 155.00
14 = 156.70 − (5.6 × 0.6) + .000005 × 560^2 = 154.91
15 = 156.70 − (6.6 × 0.6) + .000005 × 660^2 = 154 92

B. STATION
15 + 40, elev. = 152.50 + 2.45 = 154.95
16 = 158.10 − (6.4 × 0.8) + .000005 × 640^2 = 155.03
17 = 158.10 − (5.4 × 0.8) + .000005 × 540^2 = 155.24
18 = 158.10 − (4.4 × 0.8) + .000005 × 440^2 = 155.55
19 = 158.10 − (3.4 × 0.8) + .000005 × 340^2 = 155.96
20 = 158.10 − (2.4 × 0.8) + .000005 × 240^2 = 156 47
21 = 158.10 − (1.4 × 0.8) + .000005 × 140^2 = 157.08
22 = 158.10 − (0.4 × 0.8) + .000005 × 40^2 = 157.89

C. STATION
22 + 40, elev. = 152.50 + (7 × 0.8) = 158.10

In special cases it may be more convenient to note that at one-quarter of the distance from A to B the correction is $\frac{1}{16}$ of *eh*, at one-half the distance it is $\frac{1}{4}$ of *eh* and at three-fourths of the distance it is $\frac{9}{16}$ of *eh*.

CONSTRUCTION—EARTHWORK.

48. Slopes and Cross-sections. The construction of a roadbed of sufficient width which is level or nearly so, implies in general cuts and fills of various depths. An essential feature of the work is that the slopes shall be such that the banks shall not give way and disintegrate, filling up the the cuts and narrowing the tops of the fills. The rate of slope is always indicated by a ratio of which the first number indicates the horizontal distance and the second the vertical. Frequently the second number is made uniformly *one* and then the first may be a fraction or a compound number. The following slope ratios will be so indicated. The rate of slope is variable, depending on the kind of soil. When a rock cut is very hard and firm, a vertical slope may be used, although on account of seams in the rock which make slipping possible after the rock has begun to disintegrate, the rock is generally taken out until the slope will average more nearly one-fourth horizontal to one vertical. As the character of the material changes from rock to earth, the slope ratio for cut must be flattened until for good, firm, loamy soil a slope of 1:1 is proper. When first excavated earth will stand at a much steeper slope than this, but the first hard rainstorm will start the disintegration which will proceed until the slope becomes about 1:1. And as it is generally cheaper to make the required excavation during the original construction, the proper slopes should be made then. Some kinds of earthy soil, such as quicksand, require even flatter slopes. Cases have been known where a cut has not ceased to slip until it had assumed a slope of about 4:1. The proper slope for a fill composed of loose

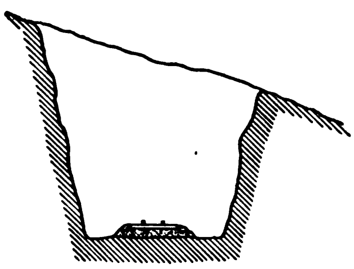


Fig. 30.

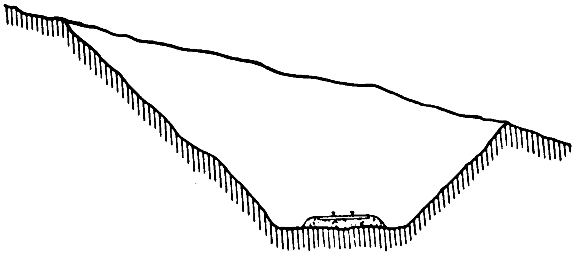


Fig. 31.

rock, as it is blasted from an excavation, is about 1: 1, but when the side hill is so steep that the slope would be very long, a much steeper slope may be tolerated if some care is taken to form the stones of the fill into a rough dry wall. A fill of earth usually requires a slope of 1.5 : 1. If the material of which the fill is made is exceptionally soft, such as would require a very flat slope in cut, then a correspondingly flatter slope should be made with the fills, but in such a case it is quite possible that it would be preferable to "waste" such treacherous soil and make the fill of more suitable soil, even if it had to be "borrowed." The following illustrations will show some typical cross-sections in various kinds of soil.

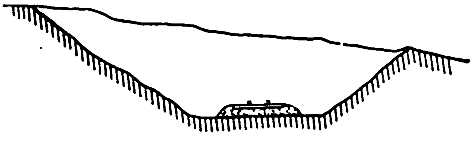


Fig. 32.

49. Width of Roadbed. A mistaken effort at economy will often be the excuse for cutting down the width of roadbed to such an amount that there is no room for adequate ditches in cuts, and the deterioration of the track due to lack of drainage is a very serious quantity. Even fills are sometimes made so narrow that the inevitable washing due to occasional heavy rains will endanger the stability of the track. A study of the standard roadbed cross-sections adopted by the principal railroads of the country shows that the average width for an earthwork cut for single track is about 25 feet. This includes about four or five feet on each side for a ditch. For double track this width is increased by about 13 feet, the usual width between track centers for two tracks. The average width for the top of a single track fill is a little over 17 feet. Sixteen feet is about the minimum for good construction.

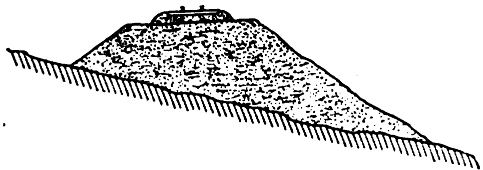


Fig. 33.

50. Constructive Details—Ditches. A well-known railroad engineer used to say that ditches were more important than ballast. A lack of good ditching will ruin a roadbed in spite of the best ballast, while a comparatively small expenditure in ditching will largely compensate on a cheap road for the lack of good ballast. The bottom of the ditch should be from one to two feet below the bottom of the ties. The slope of the sides should not be steeper than 1:1 unless in solid rock. The bottom should be one to two feet wide. Sometimes they are made V-shaped, but that shape is hydraulically bad and the bottom quickly fills up. *Sub-grade.* The upper surface of the earthwork, commonly called sub-grade, is usually made a level plane, but it is preferable to make two sloping planes having a crown at the center of about six inches. Rolling the sub-grade with a road roller before placing the ballast has been specified by the N. Y. Central R.R. When this is done, the water that runs through the ballast will more readily run off to the side ditches instead of soaking into the sub-soil. If the vegetable mould, which is usually found on the surface and which is the first of the excavation for cuts, is laid on one side instead of being placed at the bottom of the nearest fill and is afterward spread on the faces of the slopes of the cuts and fills, a growth of vegetation will quickly start up which will save the slopes from the effects of rain washing and will quickly repay the slight additional expense. Even an immediate sodding of the slopes during construction, although it will add considerably to the original cost, will usually save much more

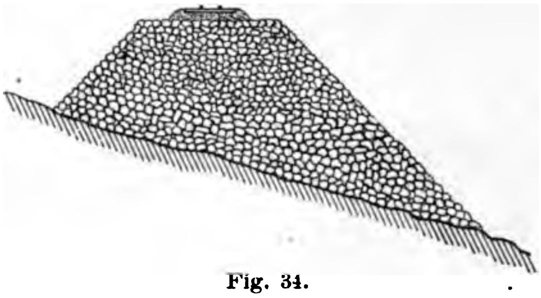


Fig. 34.

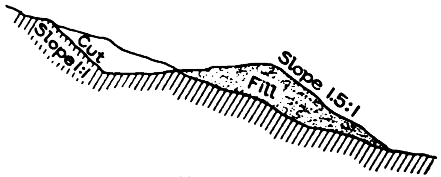


Fig. 35.

than its cost by a reduction of maintenance charges during the first few years.

EARTHWORK—SURVEYS.

51. Nature of the Problem. The mass of earthwork which is removed has an exceedingly irregular form. The surveys have the double object of staking out the limits of the earth to be removed and placed elsewhere and also of computing the volume of that earthwork. The computation of any volume means the computation of some geometrical form or combination of forms which are assumed to represent with sufficient accuracy the actual volume under consideration. If an approximate result is sufficiently close, some simple geometrical form, easily measured and easily computed, will be selected as representing the volume. But as greater accuracy is required, the more complicated must be the form. Some of the faces of the volume are simple and determinate. Sections are usually taken 100 feet apart and perhaps closer if the ground is very irregular. Such sections are plane surfaces. The side slopes are also plane surfaces. But the side representing the surface of the earth is actually rough and irregular; usually it is too irregular to be considered a plane even approximately. The upper line of each end section is considered as a broken line of one or more parts, and it is generally assumed that plane or warped surfaces connecting corresponding lines in the end sections will lie sufficiently close to the actual surface so that the error involved will be within the desired limits. It may thus be seen that the accuracy of the computation depends not only on the accuracy of the mere numerical work but even more on the judgment used by the surveyor in so selecting the places for the cross-sections and the points of any cross-section that the geometrical form assumed to represent the volume shall agree with the actual volume of earth as closely as desired. The survey therefore consists first in determining at each section the points where the side slopes intersect the surface and then the elevation and distance from the center of points so chosen that straight lines joining these points will lie very nearly in the surface of the ground.

52. Position of Slope Stakes. The slope stakes are set where the side slopes intersect the surface. As seen by the figure; these intersections depend on the elevation of the roadbed, which in turn depends on the depth of cut or fill. In Fig. 36 it may readily be seen that

$$\begin{aligned} x_l &= \frac{1}{2}b + s(d + y_l) \\ x_r &= \frac{1}{2}b + s(d - y_r) \end{aligned} \qquad (37)$$

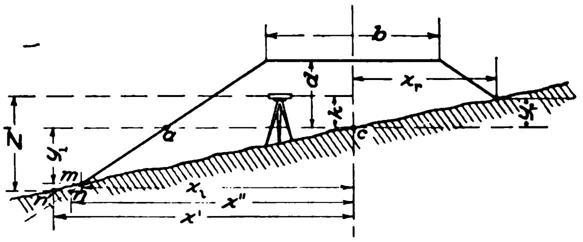


Fig. 36.

in which s is the slope ratio of the side slopes, horizontal to vertical. See §48.

Similarly it is seen in Fig. 37 that

$$\begin{aligned} x_l &= \frac{1}{2}b + s(d - y_l) \\ x_r &= \frac{1}{2}b + s(d + y_r) \end{aligned} \qquad (38)$$

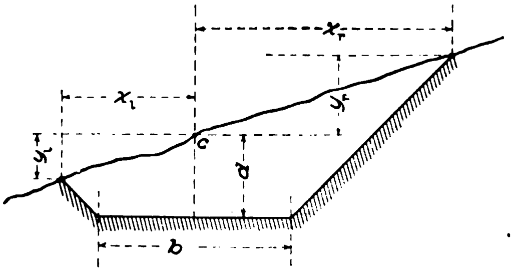


Fig. 37.

But in each of these equations, both x and y are unknown quantities, and it is impracticable to depend on a strictly mathematical solution. The simplest method is to find by trial the location of points which will satisfy the equations. An experienced man will sometimes determine such a point in a single trial and generally the second trial will be sufficient. As a first approximation, we may note that a is at such a position that its x is $ac = \frac{1}{2}b + sd$. The added distance out to m equals the added drop times s. Assume that in Fig. 36, $d = 7.7$, $b = 20$ and $s = 1.5 : 1$. The distance $ac = 10 + (1.5 \times 7.7) = 21.55$. But experience will suggest that the required point m is *about* 8 feet lower down and therefore about (1.5×8) or 12 feet further out. As a first trial with the rod, the rod is placed at n', 34 feet out $(21.55 + \text{about } 12)$, where a rod reading of z' $(= 10.6)$ is found. Subtracting k $(= 3.5)$ we have 7.1, the "y" of that point. Substituting this value of y in the first part of equation 37, we compute x_1 to be 32.2. This is the point n in Fig. 36, at a distance x'' (which is less than x') from the center. This shows that even 32.2 is too far out. Another trial is made at 30.2 feet, where the rod reading is found to be 9.3, which means that the y is $9.3 - 3.5 = 5.8$, which substituted in the equation gives $x = 30.25$. This checks so closely with 30.2, that it may be considered satisfactory. On rough ground it is an utterly useless refinement to attempt to do work closer than the nearest tenth of a foot, for the almost unavoidable inaccuracies will often have a greater effect than a change of even a tenth in such work. The above explanation is given in detail so as to show the method. Some such method is necessary for the inexperienced man, but even a short experience will enable a man to estimate the correction to his first trial very quickly and surely so that the second trial will be satisfactory, and without a detailed solution as above of all the work.

In Figs. 36 and 37, the ground has been shown as having a practically uniform slope. The determination of the slope stake is not affected essentially by the nature of the ground between the center and the slope stakes. In Fig. 42 is shown a more complicated cross-section in which the elevation of each intermediate point above the roadbed and its distance from the center must be measured. These are determined by setting up the level so that it is higher than any point in the cross-section and noting its height above the stake at the center. This rod reading added to the given center cut d gives the height of the instrument above the roadbed. This is called the H.I. Subtracting the rod reading for any point from the H.I. gives the height of that point above the roadbed. In the case of a fill, which may be illustrated by turning Fig. 42 upside down, the level may be either above or below the roadbed. This modifies the above rule somewhat, but the same principle applies—determine the difference of elevation of each point of the surface of the ground and the roadbed.

COMPUTING THE VOLUME.

53. Common Methods. Sometimes an approximate computation of the volume of the earthwork is made from the work of the preliminary survey, so as to get an approximate idea of the amount of earthwork on a route, and therefore its cost. To do this, a more or less approximate measure of each cross-section is taken and then the distance between any two cross-sections is multiplied by the half-sum of the two areas. The sum of all such products gives the total volume. Such a method is mathematically inaccurate, but the approximation in the cross-sectional areas, and some other reasons, will probably introduce still further inaccuracies. These various methods will be described in an increasing order of accuracy.

54. Level Sections. From Fig. 38 may readily be derived the equation

$$\text{Area} = (a + d)^2 s - \frac{ab}{2} \qquad (39)$$

If A_0 is the area of the initial section and $A_1, A_2, \cdot \cdot A_n$ be the areas of the succeeding and final sections, which are at a uniform distance apart of l, then the total volume will be

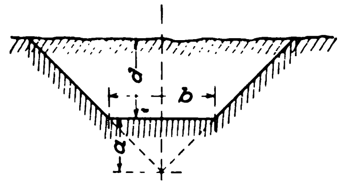


Fig. 38.

$$\text{Volume} = \frac{1}{2} l \left[A_0 + 2 (A_1 + A_2 + \cdot \cdot + A_{n-1}) + A_n\right] \qquad (40)$$

Of course l is usually 100. The $\frac{ab}{2}$ for each section is a constant, and therefore the subtractive term is simply this constant multiplied by the number of times the areas are used in the summation.

Example. Given the center heights set down in the second column of the tabular form. Width of roadbed, 20 feet; slope, 1.5 : 1. Then $d = 6.7$. The remainder of the solution is evident.

Sta.	Center Height.	(a + d)	$(a + d)^2$	$(a + d)^2 s$			
42	1.4	8.1	65.61	98.41			98.41
43	2.6	9.3	86.49	129.73			259.46
44	4.3	11.0	121.00	181.50	×	2 =	363.00
45	8.9	15.6	243.36	365.04			730.08
46	3.1	9.8	96.04	144.06			144.06

1595.01

$$\frac{ab}{2} = \frac{6.7 \times 20}{2} = 67 \qquad 8 \times 67 = 536.$$

1059.01

$$\frac{1059 \times 100}{2 \times 27} = 1961 \text{ cubic yards.}$$

The above method invariably gives results which are somewhat too high, for the volume between two "level sections" is less than the length times the mean of the areas. When the areas are equal, the error is zero, but it increases as the *square* of the difference of the center cuts, or fills. But since sections are almost never truly level, the assumption that they are level will usually introduce an error largely in excess of the theoretical error. Sometimes the above method is used, aided perhaps by tables, by taking center cuts, or fills, from a profile and assuming that the actual volume will be the equivalent of the volume computed as above. Such a method has its value as a means of comparing two routes, but the error is apt to be very great.

55. Equivalent Sections. The following method is sometimes used when the cross-sections are irregular and especially when there is disinclination or inability to use a more accurate method. Each cross-section is plotted on cross-section paper. Then a thread (mn) is so laid that (by estimation) it equalizes the spaces above and below it (see Fig. 39). The distances out from the center of the intersections of this mean line with the side slopes are scaled from the drawing and are here called x_l and x_r. Since s is the slope ratio, $s = x_l \div mo = x_r \div np$. Then the required area equals the area *mnop* minus the triangles *mso*, *nps*, and the "grade triangle," which means that

$$\text{Area} = \frac{1}{2}\left(\frac{x_l + x_r}{s}\right)(x_l + x_r) - \frac{x_l}{s}\cdot\frac{x_l}{2} - \frac{x_r}{s}\cdot\frac{x_r}{2} + \frac{ab}{2}$$

$$= \frac{x_l\,x_r}{s} - \frac{ab}{2} \qquad (41)$$

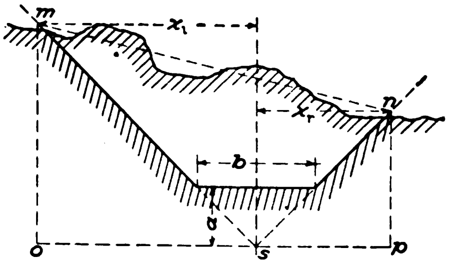


Fig. 39.

Note the simplicity of the form. When $s = 1:1$, the area equals the mere product of these two side distances minus the constant correction $\frac{1}{2}ab$. The areas being computed, the volumes are obtained exactly as in equation 40. As in the previous section, it may readily be shown that the method of averaging end areas does not give correct results except in the special case when the distances to the right (or to the left) at adjacent stations are equal, and when these distances are nearly equal the error is small. As an approximate method, it is very rapid and good. As before, the correction is usually *negative*, *i.e.*, the computed volume is too large.

56. Volume of a Prismoid. Fig. 40 represents in a perspective view a prismoid, formed between two triangles which lie in parallel planes. The surfaces which join the corresponding sides of the triangles are, in general, warped surfaces. It may be proved that the volume of such a prismoid equals one-sixth of the perpendicular distance between the parallel planes times the sum of the two end triangles and four times the middle triangle (cut by a plane parallel to the end planes and midway between them). This may be stated algebraically as follows:

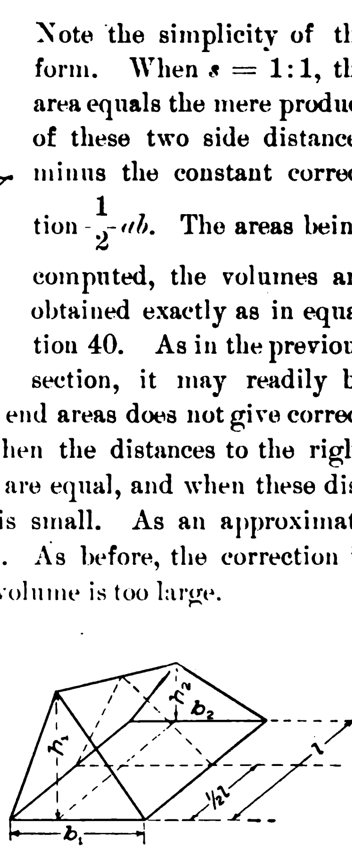


Fig. 40.

$$\text{Volume} = \frac{\text{length}}{6}\left(A_1 + 4A_m + A_2\right) \qquad (42)$$

It may also be proved that the formula holds good regardless of the values, relative or absolute, of b_1, b_2, h_1 or h_2. Therefore it holds good when $h_2 = 0$, and the prismoid becomes a wedge. It also holds good when both h_2 and b_2 become zero and the prismoid becomes a pyramid. But since the formula holds good for all these forms individually it holds good for them collectively, and since any prismoid, having bases of straight lines lying in parallel planes and with plane or warped surfaces connecting those ends, can be considered as made up of a collection of triangular prismoids, pyramids and wedges, the formula evidently holds for such a prismoid.

If, in equation 42, A_m were the mean of A_1 and A_2, then we could obtain the true volume by averaging end areas. Some of the exceptional cases where this is true have already been mentioned. In general it is a complicated and impracticable problem to compute the area of the middle section. But it is quite possible to compute the correction which must be applied to the result found by averaging end areas, and these methods will be used in the following more accurate solutions. Applying equation 42 to Fig. 40, we have

$$\text{Volume} = \frac{l}{6}\left[\frac{1}{2}b_1 h_1 + 4\left(\frac{1}{2}\,\frac{b_1 + b_2}{2}\,\frac{h_1 + h_2}{2}\right) + \frac{1}{2}b_2 h_2\right]$$

But the approximate volume, computed by averaging end areas, is

$$\text{Appr. vol.} = \frac{l}{2}\left(\frac{1}{2}b_1 h_1 + \frac{1}{2}b_2 h_2\right)$$

Subtracting the approximate volume from the true volume, we obtain the

$$\text{Correction} = -\frac{l}{12}\left[(b_1 - b_2)(h_2 - h_1)\right] \qquad (43)$$

57. Three-Level Sections. When the ground is fairly uniform so that it may be said without great inaccuracy that it slopes uniformly from the center to each slope stake, then the volume may be computed from the positions of these three points and the sections are called "three-level sections." The area of such a section $= \frac{1}{2}(a + d)\,w - \frac{1}{2}ab$. If we consider two such adjacent sections and compute the volume by the method of averaging end areas, we will obtain as the volume

$$\text{Vol.} = \frac{l}{4}\Big[(a + d')\,w' - ab + (a + d'')\,w'' - ab\Big]$$

Dividing by 27 to reduce immediately to cubic yards, we have when $l = 100$,

$$\text{Vol.} = \frac{25}{27}(a + d')\,w' - \frac{25}{27}ab + \frac{25}{27}(a + d'')\,w'' - \frac{25}{27}ab \qquad \textbf{(44)}$$

When it is desired to make the computation still more accurate, the prismoidal correction may be computed as follows. We may compute separately the prismoidal correction for each of the two triangular prismoids. These two prismoids together include the triangular "grade prismoid" under the roadbed, but since there is no prismoidal correction to the grade prismoid, such correction as may be computed applies solely to the volume actually excavated. Applying equation 43 to the dimensions in Fig. 41, we have for the left side

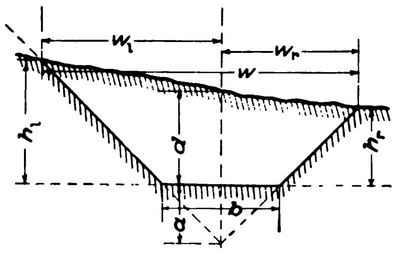


Fig. 41.

$$\text{Pris. Corr.} = \frac{l}{12}\Big[(a + d') - (a + d'')\Big](w_l'' - w_l'),\ \text{which}$$

$$\text{reduces to} - \frac{l}{12}(d' - d'')(w_l'' - w_l')$$

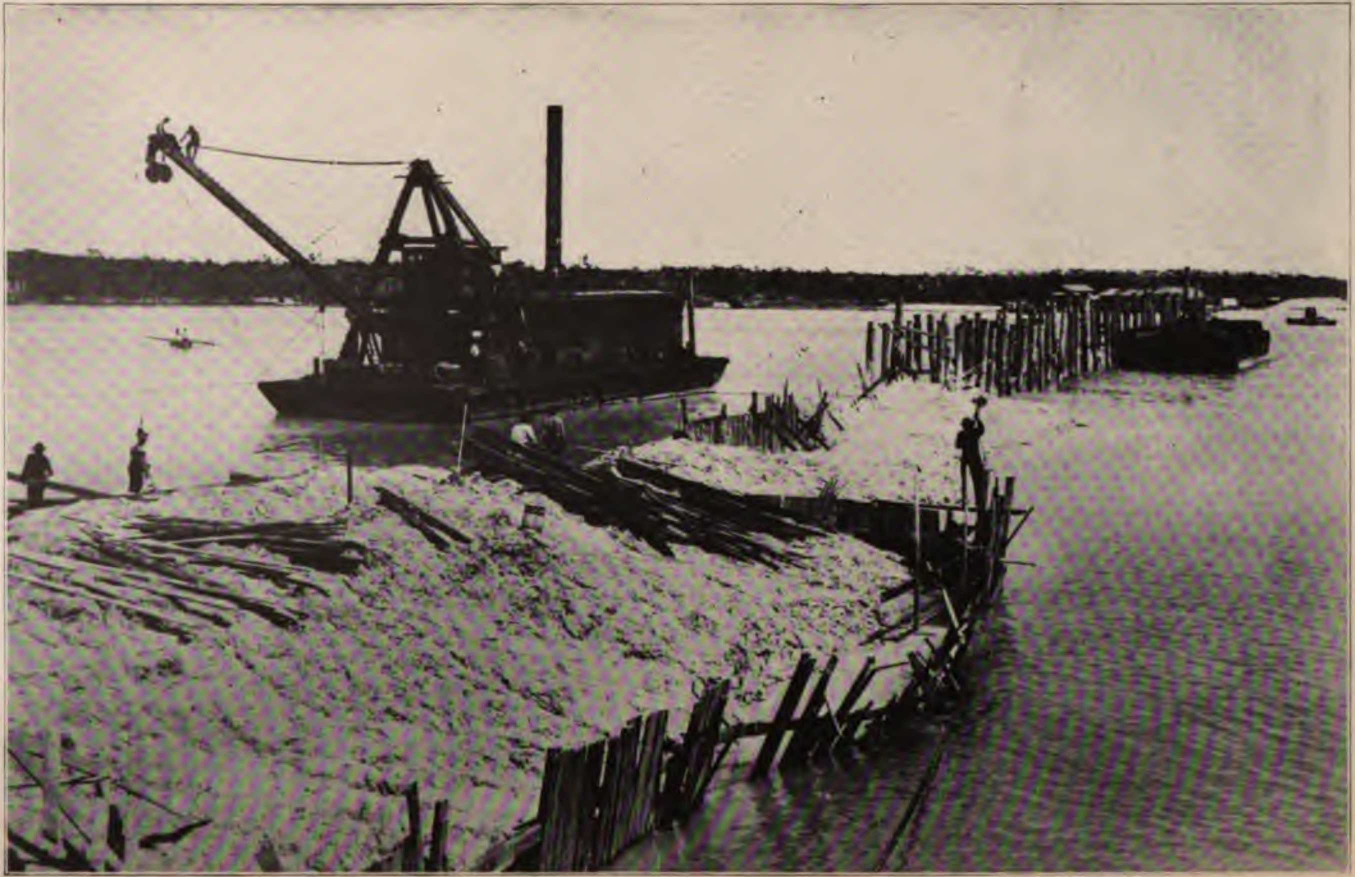
For the right side, we may compute similarly

$$\text{Pris. Corr.} = -\frac{l}{12}(d' - d'')(w_r'' - w_r')$$

For the two triangles we have

$$\text{Pris. Corr.} = -\frac{l}{12}(d' - d'')\Big[(w_l'' + w_r'') - (w_l' + w_r')\Big]$$

$$= -\frac{l}{12}(d' - d'')(w'' - w')$$



RAILROAD BUILDING OUT TO SEA

Construction of a four-mile fill between Lower Matacumbe and Long Keys, in the extension of the Florida East Coast Railway to Key West.

Making $l = 100$ and dividing by 27 to reduce to cubic yards we have

$$\text{Pris. Corr.} = \frac{25}{81}(d' - d'')(w'' - w') \qquad (45)$$

An inspection of equation 45 will show that if either the center cuts or the total widths at two adjacent sections are equal or nearly so, the prismoidal correction is zero or is so small that it may be neglected. This frequently enables one to decide that the prismoidal correction will evidently be so small that it will be useless to compute its exact value. It usually happens that when $d' > d''$, w' is also greater than w''. This means that the correction computed from equation 45 will usually be *negative*, which means that for three-level sections the results computed by averaging end areas will usually be too large.

A very great economy of time and accuracy result from tabulating all the computations in earthwork. Such work can be readily reviewed to check it or to discover a supposed error. An illustration of such a solution is given below.

58. Numerical Example.

Sta.	Center.	Left.	Right.	a + d	w	Yards		d' d''	w'' w'	Pris. corr.
52	3.1 C	9.6 C 26.4	1.2 C 13.8	11.1	40.2	413				
53	6.7 C	11.4 C 29.1	4.2 C 18.3	14.7	47.4	645	702	-3.6	+7.2	8
54	10.6 C	15.6 C 35.4	7.8 C 23.7	18.6	59.1	1018	1307	3.9	+7.7	9
+65	15.5 C	19.0 C 40.5	10.6 C 27.9	23.5	68.4	1488	1397	4.9	+9.3	9
55	8.7 C	12.4 C 30.6	5.8 C 20.7	16.7	51.3	793	674	+6.8	17.1	13

Roadbed 24′ wide in cut. Approx. vol. - 4080 -- 39

Slope 1.5 : 1. Pris. corr. - 39

$$a = \frac{b}{2s} = \frac{24}{3} = 8.0 \qquad \text{True vol.} - 4041$$

$$\frac{25}{27}ab = 178$$

In the above form, $\frac{9.6\,C}{26.4}$ in the third column means that the slope stake on the *left* side of Sta. 52 is 9.6 feet above the elevation of the roadbed which is here in *cut* C; also that it is 26.4 feet out from the center. This notation is also used to indicate the position of "intermediate points," the numerator of the fraction giving the depth of cut or fill (C or F) and the denominator the distance from the center. The other points in the third and fourth columns are to be interpreted similarly. Column 5 is found by adding a $(=8.0)$ to column 2; w in each case is the sum of the two denominators in the same horizontal line; 413 (in column 6) $= \frac{25}{27} \times 11.1 \times 40.2$. A short method of performing this multiplication will be given later. The solution of equation 44 applied to this case is:

$$\text{Vol.} = 413 - 178 + 645 - 178 = 702.$$

Similarly $1397 = \frac{65}{100}(1018 - 178 + 1488 - 178)$.

and $674 = \frac{35}{100}(1488 - 178 + 793 - 178)$.

$-3.6 = 3.1 - 6.7$ and $+7.2 = 47.4 - 40.2$. $-8 = \frac{25}{81}$ $(-3.6) \times (+7.2)$; see equation 45. Note that in this case the prismoidal correction is less than 1 per cent of the total volume. The errors due to inaccurate cross-sectioning will frequently be more than this. The volume 4041 cubic yards is the *precise* volume (barring the neglect of the fraction of a yard) of the prismoids given by the notes. Whether these prismoids actually represent the true volume of the earthwork depends entirely on the cross-sectioning and is entirely out of the hands of the computer.

59. Computation of Products. The products $\frac{25}{27}ab$ may be written $\frac{ab}{1.08}$. These products are always the combination of two variable terms and a constant. It thus becomes possible to construct tables which will give these products for any given height and width. Crandall's Earthwork Tables are computed on this basis. But these products are also obtained with great ease by means of a slide rule, provided it is large enough to give the required accuracy. The 108 mark, being so constantly in use should have a special mark so that it may be found without effort.

As a numerical illustration, take the first of the above cases. Set the 108 mark of the upper scale on the 111 mark on the lower scale. Then look for the 402 mark on the upper scale and note that it is nearly over the 413 mark on the lower scale. While it is possible to devise set rules to determine the position of the decimal point, it is considered that a hasty mental solution of the problem will decide the point quicker and with less chance of error. For example—the product of the two variable quantities is always divided by 1.08, which means that the final result will be a little less than the simple product of the two variables, 11 × 40 = 440. Therefore 413 is evidently the correct result, rather than 41.3 or 4130. The products $\frac{25}{81}xy$ are similarly obtained since $\frac{25}{81} = \frac{1}{3.24}$, and the 324 mark can be used instead of the 108. For example, the slide rule shows that $\frac{(-3.6) \times (+7.2)}{3.24} = -8$ to the nearest cubic yard. As to the decimal point—3.6 ÷ 3.24 is something more than one; therefore, the result is something more than 7.2. Therefore it is 8, rather than 80 or 0.8. If the student has neither tables nor slide rule, the multiplication of the two variables (in columns 5 and 6) and the division of the products by the constant 1.08, may be made so mechanical and systematic that it may be done quickly and accurately although it is much slower than the slide rule method.

60. Irregular Sections. The distance from the center and the height above or below the roadbed must be obtained for each break in the surface between the slope stakes. Then, in Fig. 42, by dropping perpendiculars from each point to the roadbed line, the total area is divided into a number of trapezoids, the sum of the areas of which (less the areas of the two triangles under the side slopes) equals the total area of the section. For Fig. 42, the area would be stated algebraically as follows:

$$\begin{aligned}\text{Area} = {} & \frac{1}{2}(r + s)(f' - g) + \frac{1}{2}(s + t)(g - h) + \frac{1}{2}(t + d)h \\ & + \frac{1}{2}(d + v)j + \frac{1}{2}(v + w)(k - j) - \frac{1}{2}w\left(k - \frac{1}{2}b\right) \\ & - \frac{1}{2}r\left(f' - \frac{1}{2}b\right).\end{aligned}$$

Expanding this and collecting terms, of which many will cancel out, we have

$$\text{Area} = \frac{1}{2}\left[fs + g(t - r) + h(d - s) + kv + j(d - w) + \frac{1}{2}b(r + w)\right]. \qquad (46)$$

Although the above equation looks as if it applied only to the particular case given, yet a little study of it will show that the terms follow a law so general that the *reduced* equation for the area of *any* section, no matter how complicated or how many points it may have, may be written out by a literal obedience of the following rule:

Area equals one-half the sum of products obtained as follows:

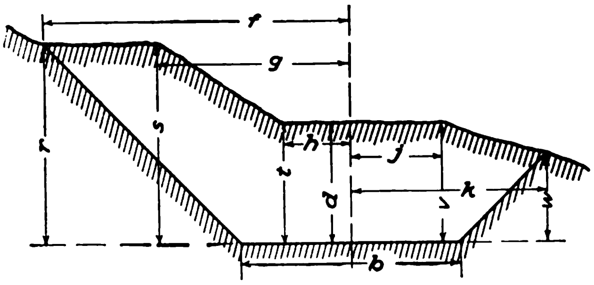


Fig. 42.

The distance to each slope stake times the height above grade of the point next inside the slope stake;

The distance to each intermediate point in turn times the height of the point just inside minus the height of the point just outside;

Finally, one-half the width of the roadbed times the sum of the slope stake heights.

The above rule may be followed literally whether there are forty intermediate points or one, or even *none*. When there are no intermediate points the terms for that side reduce to one—the distance to the slope stake times the height of the point next inside (which in this case is the center). For three-level ground (see Fig. 41), we would have three terms;

Area $= w_l\,d + w_r\,d + \frac{1}{2}\,b\,(h_l + h_r)$, which reduces to two terms

Area $= wd + \frac{1}{2}\,b\,(h_l + h_r)$.

The method of § 57 has the advantage that one of the two terms for each section is constant for all sections—in the numerical case of § 58 it is 178 cubic yards. By the above method the two terms for each section are variable. Nevertheless, when the cross sections usually have one or more intermediate points and therefore the method of § 60 must be used, and an occasional section is found with no intermediate points, it is better for the sake of uniformity to apply the above method rigidly and thereby avoid the possible confusion and error that would result by the use of another method. Probably no time would be saved by the change of method.

61. Prismoidal Correction. The above method of irregular sections is capable of being followed to its logical conclusion, and a computation for volume made which is mathematically correct, provided that it is noted on the ground how the points of adjacent sections are joined, so that the warped surfaces thereby developed shall lie as closely as possible in the actual surface of the ground. But the very fact that the cross-section is irregular implies that any longitudinal section will be correspondingly irregular, and this leads to the suspicion that a refinement in the computations may be overshadowed by a much larger but unknown difference between the volume of the assumed geometrical solid and the actual volume of the earthwork. In order to obtain a correction which is easily computed and which gives a result which is probably much nearer the truth than no correction at all, the following method is much used: Consider that *for the purpose of the correction* the ground is "three-level ground" and use equation 45. Numerical computations of volumes by both methods have shown that the *difference* is small, and is perhaps smaller than the probable error on the entire work. This method will be used in the following numerical solution:

62. Numerical Example. Volume of Earthwork in Irregular Ground. The first tabular form gives a desirable form of notes for recording the field work. Note that the station numbers

Sta.	Center Cut or Fill.	Left.			Right.	
47	1.2 c	$\frac{4.2\text{ c}}{16.4}$				$\frac{0.8\text{ c}}{11.2}$
46	4.3 c	$\frac{5.1\text{ c}}{17.6}$	$\frac{6.2\text{ c}}{8.5}$		$\frac{2.1\text{ c}}{3.4}$	$\frac{1.6\text{ c}}{12.4}$
+ 42	13.6 c	$\frac{20.2\text{ c}}{40.3}$	$\frac{15.7\text{ c}}{32.4}$	$\frac{14.4\text{ c}}{16.8}$	$\frac{12.5\text{ c}}{10.2}$	$\frac{9.6\text{ c}}{24.4}$
45	9.2 c	$\frac{12.3\text{ c}}{28.5}$	$\frac{12.7\text{ c}}{16.0}$	$\frac{6.8\text{ c}}{6.4}$	$\frac{9.2\text{ c}}{8.5}$	$\frac{7.8\text{ c}}{21.7}$
44	3.2 c	$\frac{6.0\text{ c}}{19.0}$	$\frac{3.5\text{ c}}{8.3}$			$\frac{1.8\text{ c}}{12.7}$

Roadbed 20 feet wide in cut. Slope 1.5 : 1.

FORM FOR REDUCING THE ABOVE FIELD NOTES.

Sta.	Width.	Height.	Yards.		Center height.	Total width.	$d' - d''$	$w'' - w'$	Approx. pris. corr.
44	19.0	3.5	62		3.2	31.7			
	8.3	- 2.8	- 22						
	12.7	3.2	38						
	10.	7.8	72						
45	28.5	12.7	335		9.2	50.2	- 6.0	+18.5	- 34
	16.0	- 5.5	- 81						
	6.4	3.5	- 21						
	21.7	9.2	185						
	8.5	1.4	11						
	10.	20.1	186	765					
+ 42	40.3	15.7	585		13.6	64.7	- 4.4	+14.5	- 20
	32.4	- 5.8	- 174						
	16.8	- 2.1	- 33						
	24.4	12.5	282						
	10.2	4.0	38						
	10.	29.8	276	667					
46	17.6	6.2	101		4.3	30.0	+9.3	- 34.7	- 100
	8.5	- 0.8	- 6						
	12.4	2.1	24						
	3.4	2.7	8						
	10.	6.7	62	675					
47	16.4	1.2	18		1.2	27.6	+3.1	- 2.4	- 2
	11.2	1.2	12						
	10.	5.0	46	265					

Approx. volume = 2372 — - 156
" pr. corr. = -156
Corrected volume = 2216 cubic yards.

run up the page. By this method the "fractions" which show the height and distance out of each point are recorded in their approximate relative position when the note book is held looking ahead along the line.

It should be noted in this case that the prismoidal correction is a large percentage of the total volume. In the case of a pyramid, the correction is one-third of the nominal volume, which means that it is 50 per cent of the true volume. The foregoing numerical case gives the notes for an embankment crossing a gully, and in such cases especially the prismoidal correction is always large and should not be neglected.

63. Side-hill Work. A road frequently runs along the slope of a hill so that it is necessary to have both cut and fill in the same section. The profile at such a place will indicate little or no earthwork, but if the natural slope is steep and the roadbed wide the amount of earthwork may be considerable. Although it is possible to apply the general rule of §60 to such cases, it is usually simpler to compute the area in each case independent of the rule, especially when the section forms a mere triangle. The areas of cut and fill must be calculated independently. When a section of cut or fill is found at one station and is not found at the next, accuracy requires a knowledge of the place where the cut or fill "runs out". That small volume must then be considered a pyramid with a given base and height. In general the end of every cut or fill implies the existence, at least for a short distance, of side-hill work. Although the volumes are usually small, yet since they are frequently of pyramidal form, the prismoidal correction is usually a large percentage of the volume and hence should not be neglected. The rule of §61 can generally be applied.

64. Borrow Pits. This name is applied to places from which earth is taken to make an embankment when there is insufficient excavated material in the neighborhood or when the material is unsuitable for embankments. Such volumes must be measured up and paid for the same as other excavation. Sometimes the form of the excavation is literally that of a rectangular pit, in which case the simple product of the three dimensions gives the volume. But usually it is required to slope the sides; sometimes the material is obtained by widening a cut, as in Fig. 44. If the borrow pit is very large, several cross-sections should be taken at sufficiently close intervals. If the prismoids into which the total volume is divided have substantially equal bases, the prismoidal correction will be small and may be neglected, but if the forms are pyramidal the correction should be computed It may become necessary to consider the total volume as made up of triangular prismoids and compute the prismoidal correction for each one separately.

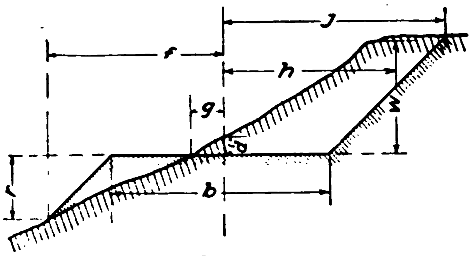


Fig. 43.

65. Correction for Curvature. The volume of a solid, generated by revolving a plane area about an axis lying in the plane but outside of the area, equals the product of the given area times the length of the path of the center of gravity of the area. When the centers of gravity of the cross-sections lie in the center of the road, where the length of the road is measured, no correction is necessary. If all the cross-sections were the same and therefore had the same eccentricity, the total volume could be computed by the above rule. But in general both the areas and the eccentricities vary from point to point, and then a theoretically exact solution would be impracticable for ordinary work if not impossible. But a sufficiently practical rule can be developed as follows: Assume a curved prismoid, of uniform cross-sections and therefore of uniform eccentricity. Call that eccentricity e. Let R be the radius of the center line of the road. Then the length of the path of the center of gravity is to the length measured on the center line as $R \pm e : R$. Therefore we have

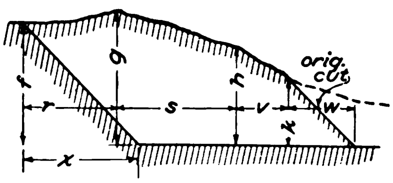


Fig. 44.

$$\textit{True volume} : \textit{nominal volume} : : R \pm e : R$$

Therefore the true volume of the prismoid $= lA\frac{R \pm e}{R}$. This shows that the effect of curvature is the same as increasing or diminishing the area by an amount which depends on the area and the eccentricity and that the increased or diminished area may be found by multiplying the actual area by the ratio $\frac{R \pm e}{R}$. This being independent of the value of l, it is true for infinitesimal lengths. If the eccentricity is assumed to vary uniformly between two sections, the *equivalent* area of a cross-section located midway between the two end areas would be $A_m \frac{R \pm \frac{1}{2}(e' + e'')}{R}$. Therefore the volume of a solid which, when straight, would be $\frac{1}{6}l$ $(A' + 4A_m + A'')$ would then become

True volume =

$$\frac{l}{6R}\left[A'(R \pm e') + 4A_m\left[R \pm \frac{1}{2}(e' + e'')\right] + A''(R + e'')\right]$$

Subtracting the nominal volume, which is the true volume when the prismoid is straight, we have the correction for curvature as follows

$$\text{Correction} = \pm\frac{l}{6R}\left[(A' + 2A_m)e' + (2A_m + A'')e''\right]$$

As in the case of the prismoidal formula, the use of this equation implies a knowledge of the middle area. This correction is always a small proportion of the total volume and is frequently insignificant. Therefore no appreciable error is involved in making the equation read

$$\text{Curv. corr.} = \frac{l}{2R}(A'e' + A''e'') \qquad (47)$$

66. Eccentricity of the Center of Gravity. *The value of "e".* The determination of the center of gravity of a complicated irregular cross-section would be a long and tedious problem and is practically unnecessary. For the purpose of this correction it is sufficiently accurate to consider all sections as three-level sections, except in side-hill work, where they should usually be considered as triangles. In Fig. 45, the eccentricity of the center of gravity of the whole section, including the grade triangle, may be computed as follows:

$$e = \frac{\frac{(a+d)\,x_l}{2}\,\frac{x_l}{3} - \frac{(a+d)\,x_r}{2}\,\frac{x_r}{3}}{\frac{(a+d)\,x_l}{2} + \frac{(a+d)\,x_r}{2}} = \frac{1}{3}\,\frac{(x_l^2 - x_r^2)}{(x_l + x_r)} = \frac{1}{3}\,(x_l - x_r) \qquad (48)$$

Substituting this value of e in equation 47, we have

$$\text{Curv. corr.} = \frac{l}{6R}\Big[A'\,(x_l' - x_r') + A''\,(x_l'' - x_r'')\Big]$$

But the approximate volume of a prismoid may be written

$$\text{Vol.} = \frac{l}{2}\,(A' + A'') = \frac{l}{2}\,A' + \frac{l}{2}\,A'' = V' + V''$$

in which V′ and V″ represent the number of cubic yards corresponding to the area at each station. Substituting these values in the above equation, we have

$$\text{Curv. corr. in cu. yd.} = \frac{1}{3R}\Big[V'\,(x_l' - x_r') + V''\,(x_l'' - x_r'')\Big] \qquad (49)$$

The value of e, found in equation 48, is the eccentricity of the whole area, including the grade triangle under the roadbed. The eccentricity of the true area is greater than this and equals

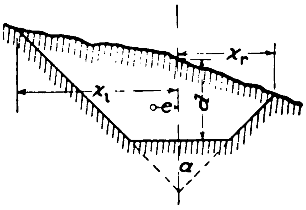


Fig. 45.

$$e^1 = e \times \frac{\text{true area} + \frac{1}{2}\,ab}{\text{true area}}$$

The required quantity (the A′ e' of equation 47) equals *true area* × e_1 which equals (*true area* $+ \frac{1}{2}\,ab$) × e. The value of e (given in equation 48) is a remarkably simple term, while the value of e_1 is very complicated and difficult to compute. But since we may obtain the true corrective value by using e and at the same time adding the yardage corresponding to the grade triangle to the yardage corresponding to each area, it is best to do it in this way.

For any one curve the corrective terms are all divided by the quantity 3R. If tables are not at hand, it is amply accurate to compute that $R = \frac{5730}{D}$ in which D is the degree of the curve. Even equation 49 may be simplified somewhat in actual use. The correction at each station has the form $\frac{V(x_l - x_r)}{3R}$. 3R is usually a large quantity; for a 4° curve it is 4298. $(x_l - x_r)$ is relatively very small. Their ratio times V is never a very large quantity, and it is frequently less than unity, when, of course, it should be ignored. An approximate solution will generally show that the product $V(x_l - x_r)$ is roughly twice or three times 3R, or is perhaps less than one-half 3R, and then the corrective term for that station may be written 3, 2, or 0 cubic yards, the fraction being ignored. It is only when the curvature is excessively sharp and the eccentricity very great that the curvature correction becomes a large percentage of the volume.

The algebraic sign of the correction is most surely and easily noted from a consideration of the direction of the curvature and on which side the earthwork predominates. If the center of gravity is evidently toward the *inside* of the curve the true volume is evidently *less* than the nominal volume and the correction should be *negative*. When the curve turns to the *right*, use the form $(x_l - x_r)$; when it turns to the *left*, use the form $(x_r - x_l)$. The algebraic sign of the correction will then be strictly in accordance with its true value.

67. Numerical Example. Assume that the earthwork computed in § 58 is located on a 10° curve to the *left*. How much will be the curvature correction? Copying from the solution in § 58 the necessary data we have at once the first four columns of the tabular form. Usually the last three columns will be merely added to the form given in § 58. Since the curve is to the left, we use the form $(x_r - x_l)$. $3R = 3 \times 573 = 1719$.

Station.	x_l	x_r	Yards.	$x_r - x_l$	$\frac{V(x_r - x_l)}{3R}$	Curv. corr.
52	26.4	13.8	413	- 12.6	- 3	
53	29.1	18.3	645	- 10.8	- 4	- 7
54	35.4	23.7	1018	- 11.7	- 7	- 11
+ 65	40.5	27.9	1488	- 12.6	- 19	- 17
55	30.6	20.7	793	- 9.9	- 5	- 8

Total curv. corr. = - 43 yards.

The net volume of the mass under consideration is thus reduced to 3,998 cubic yards. A 10° curve is rather unusual. Since the correction varies directly as the degree of the curve, if the above curve were a 4° curve the correction would be only 0.4 × 43 = 17 cubic yards.

68. Eccentricity of the Center of Gravity of a Side-hill Section. It will generally be sufficiently accurate to consider, for this purpose, that all side-hill sections are triangular. The center of gravity of a triangle lies on a line joining the vertex and the middle of its base, and at one-third of the length of this line from the base. The eccentricity is therefore equal the distance from the center to the middle of the base of the triangle plus one-third of the base of the triangle plus one-third of the horizontal projection of that line. Applying this rule to Fig. 46, we have

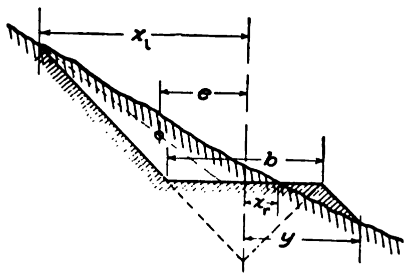


Fig. 46.

$$e = \left[\frac{b}{2} - \frac{1}{2}\left(\frac{b}{2} - x_r\right)\right] + \frac{1}{3}\left[x_l - \left(\frac{b}{2} - \frac{1}{2}\left(\frac{b}{2} \times x_r\right)\right)\right]$$

$$= \frac{b}{4} - \frac{x_r}{2} + \frac{x_l}{3} - \frac{b}{12} + \frac{x_r}{6}$$

$$= \frac{b}{6} + \frac{x_l}{3} - \frac{x_r}{3}$$

$$= \frac{1}{3}\left[\frac{b}{2} + (x_l - x_r)\right] \qquad (50)$$

It should be noted that the grade triangle is not considered in the above solution ; therefore the volume of the grade prism should not be considered in applying Eq. 49. In three-level ground the curvature correction will be zero when $x_l = x_r$, but in side-hill work the curvature correction is never zero and it may be a large proportion of the total volume.

In case the triangle lies wholly on one side of the center, it may be similarly shown that the equation may be written

$$e = \frac{1}{3}\left[\frac{b}{2} + (x_l + x_r)\right] \tag{51}$$

This equation may be derived directly from equation 50 by considering that when both points are on the same side of the center, the algebraic sign x_r should be changed. These various cases may be generalized by saying that when the triangle lies on *both* sides of the center, e is always numerically equal to $\frac{1}{3}\left[\frac{b}{2} + (x_l \sim x_r)\right]$ in which the form should be $(x_l - x_r)$ or $(x_r - x_l)$ according to the criterion used in § 66. When the triangle is on one side only $e = \frac{1}{3}\left[\frac{b}{2} + \text{the numerical } \textit{sum} \text{ of the two distances out}\right]$. Its algebraic sign should be determined as in § 66.

CONSTRUCTIVE EARTHWORK.

69. Methods of Excavating. Economical excavating depends largely on the distance to be hauled as well as on the character of the soil. Side-hill work is usually done by mere shoveling, the earth being loosened by picks or plows. When the distance that the earth must be moved becomes greater, wheelbarrows or drag scrapers become economical. Wheeled scrapers, two-wheeled carts, four-wheeled wagons, small cars drawn by horses, and heavier cars drawn in a train load by locomotives successively become more economical as the distance increases. As the magnitude of the work increases, thereby justifying an increase in the cost of the plant used, economy in the cost of loosening and loading is obtained by using a steam-shovel instead of picks and plows. When cuts are very deep, they are best excavated in "benches" whose height will depend on the method of loosening and loading.

70. Blasting. Blasting is employed not only for hard rock which can only be removed by such methods, but also as a means for rapidly loosening shale and even frozen earth. The explosives used vary in their composition from a high grade detonating explosive, such as No. 1 dynamite which consists of 75 per cent nitro-glycerine, to black powder which is, comparatively, "slow burning." Between these extremes there are a great multitude of explosive compounds, which consist of varying proportions of explosives of the two types. It has been demonstrated that a slow burning explosive is made to "detonate" if it is exploded by means of a sufficient volume of a detonating explosive, which means that a mixture of the two kinds has a greater explosive force than the sum of the constituents exploded separately. The choice of explosive depends on the character of the rock. A hard brittle rock requires a detonating explosive which shall shatter it. A soft and tough rock is best loosened by a powder which acts more slowly. If dynamite is exploded in a soft clay rock the hole may be blown out, but the great mass of the rock is not disturbed. When the center of the mass of powder is 4 feet from the nearest surface, about 2 pounds of black powder (or about $\frac{1}{3}$ of a pound of dynamite) should be used. The amount should vary as the *cube* of the "line of least resistance," *i.e.*, if the center of the blast is 10 feet from the nearest surface the amount would be determined by the proportion $x : 2 :: 10^3 : 4^3$, or $x = 31$ pounds. In this case, since dynamite is about six times as powerful as black powder, a little over 5 pounds of No. 1 dynamite would do as well. The "line of least resistance" may not be the "nearest line to the surface" on account of seams in the rock which modify its resisting power, but the above rule is about as near a fixed rule as can be stated. For open work, especially when time is not a very important matter, it is cheaper to use black powder, but in tunnel headings where the progress of the work is limited by the progress of the drillers it is economical to use dynamite although it is more expensive.

Drilling. Hand drilling when the holes are vertical is best accomplished with "churn drills," which are heavy bars of iron shod with a steel drill, which are raised and dropped, the impact doing the cutting, the drill being slightly turned after each stroke. From five to fifteen feet of holes, depending on the character of the rock, is considered a fair day's work of ten hours. Oblique or horizontal holes must be drilled with light drills of the "one-man" type or the "two-man" type. The one-man drill is worked entirely by one man with a light one-hand hammer. With the other method, one man holds the drill, which is perhaps a little heavier and which is struck by another man, or perhaps by two, using a heavy hammer. It has been found that the light-hammer method is more economical for soft rocks, the heavy-hammer method more economical for hard rocks, but that the light-hammer method is always quicker and is to be preferred when limited space and time are matters of importance. Machine drilling is a specialty which can only have a very brief general discussion here. Where the magnitude of the work will justify it, it is always more economical per foot of hole drilled. The plant is expensive both in first cost and maintenance; part of the expense is nearly constant regardless of the number of holes bored, and so it is only when the work is extensive that the method is advantageous, but under favorable circumstances the economy is very marked. For open-pit work individual drills are used, but for tunnel headings several drills are mounted on a "carriage" from which, after it is set, several holes may be drilled simultaneously. Compressed air is used to run the drills in tunnel work. This serves the additional purpose of furnishing a supply of pure cold air at the place where it is most needed.

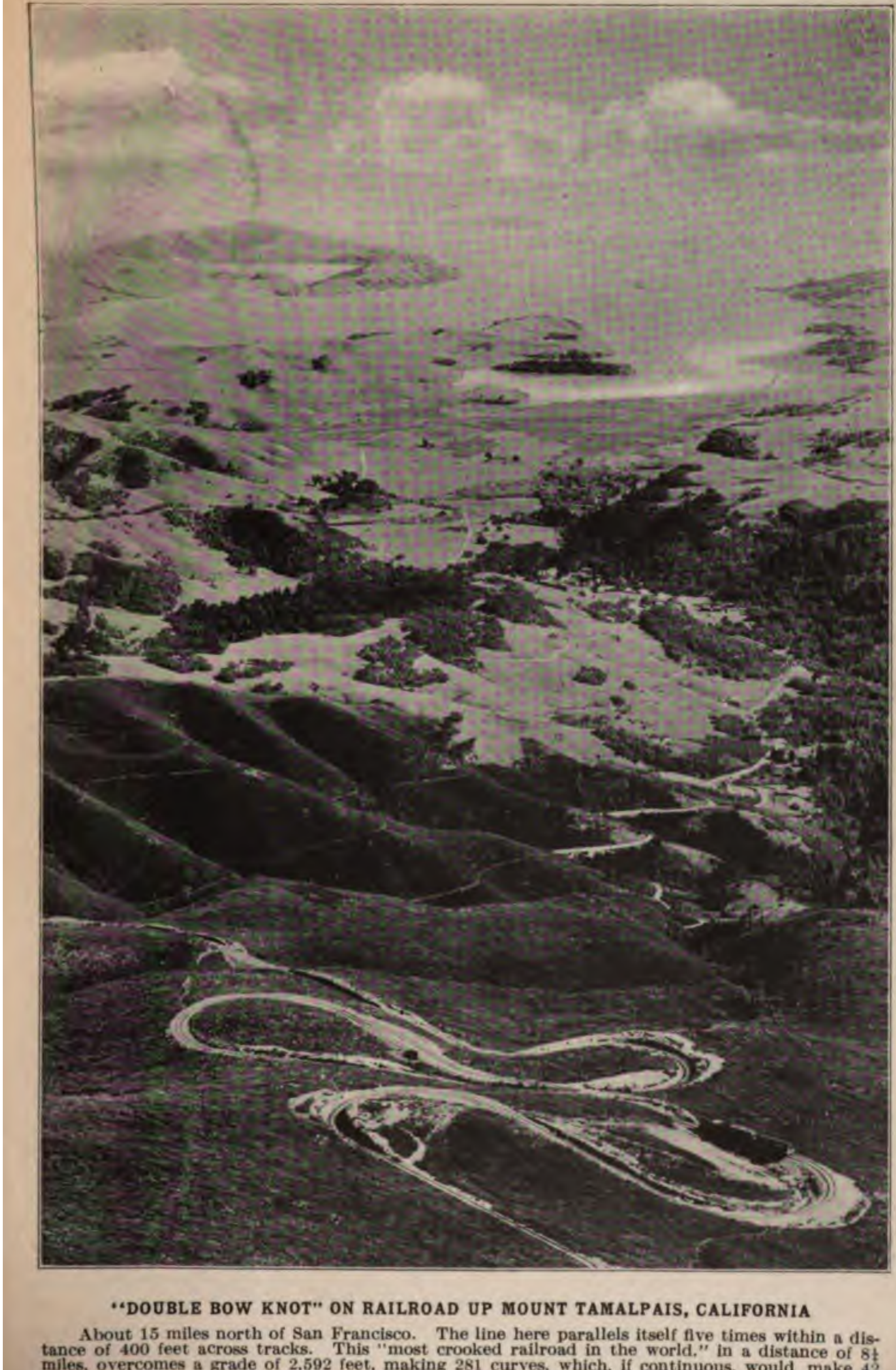
Tamping. It has been found that air spaces around the explosive cause a very material reduction in the force of the explosion, therefore it is necessary to ram the explosive into a solid mass and then pack the top of the hole. Iron bars should never be used for tamping. Copper bars are sometimes used for ramming powder, but dynamite is most safely rammed with wooden bars. Clay is the best tamping material where it is available, but sand or finely powdered rock will serve very well. It has been found that when blasting under water the weight of the superincumbent water is sufficient tamping.

Exploding. On small-scale work, the blasts are generally exploded by means of a powder fuse, which is essentially a cord which forms the matrix for a train of powder, the cord being further protected by a wrapping of some sort. The better plan, and the one which is used almost exclusively for extensive work, is to explode a large number of holes simultaneously by means of electricity. A "cap" containing a small charge of fulminate of mercury, an expensive but very powerful explosive, is set in the midst of the larger mass of explosive. An electric current from a field battery is sent through each cap. As the current passes through each cap it heats a small platinum wire to redness or else causes a spark to jump across a short gap in the wire. In either case the fulminate is exploded, which in turn explodes the main charge.

Cost. The cost of blasting is so exceedingly variable, depending on the nature of the rock, depth of the cutting, and especially on the magnitude of the work and the methods employed, that only the most approximate estimates can be here given. Under the most favorable circumstances, deep cutting, machine methods, and a rock which is brittle but not too tough, the cost may fall as low as 25 cents per cubic yard, while with hand drilling, hard and tough rock, in a shallow cut, the cost might easily rise to $1 per cubic yard. It would indicate exceptionally unfavorable circumstances, bad management, or possibly an excessive price for common labor, if the cost should rise above this figure.

71. Formation of Embankments. Experience has shown that when earth is excavated and piled in embankments its volume will at first be more than the original measured volume but that it will finally shrink to about 90 per cent of its original volume. The percentage of shrinkage is a very uncertain quantity, as it depends on the kind of earth, on the method employed in forming the embankment and on the time elapsed between construction and the measurement of what is supposed to be the settled volume. Material dumped from a trestle will first have a volume considerably in excess of its final volume and it will take several months and even years to shrink to its final volume. On the other hand, if an embankment is formed in very thin layers, each of which is packed down by the process of unloading the succeeding layer, there will be but little shrinkage after the embankment is finished, but more material than the volume as measured in the cut will be required to form that volume of embankment. Broken rock, formed into an embankment, will have a volume about 80 per cent greater than the mass of solid rock from which it is taken.

It is frequently specified that embankments are to be made to a somewhat higher elevation than the plan of the road calls for, so that the expected shrinkage will reduce the embankment to the desired level. Since the contractor is paid by the cubic yards of material *excavated* and is required to dispose of excavated material as required, it is generally specified that the amount of this



"DOUBLE BOW KNOT" ON RAILROAD UP MOUNT TAMALPAIS, CALIFORNIA

About 15 miles north of San Francisco. The line here parallels itself five times within a distance of 400 feet across tracks. This "most crooked railroad in the world," in a distance of 8¼ miles, overcomes a grade of 2,592 feet, making 281 curves, which, if continuous, would make 42 complete circles. The locomotives used are oil-burners of the "marine-geared" type, with three vertical cylinders on one side operating the drivers by turning a long, horizontal shaft with cogged gearing.

excess of height of embankments is left to the discretion of the engineer who decides the question during the progress of the work and after he has had an opportunity to judge of the material after excavation is under way. When embankments are placed on sidehills, the surface should be first plowed or have trenches dug along the slope so that the embankment shall not slip down the hill. A ditch dug at the base of each slope will drain the subsoil and may prevent a dangerous and costly disintegration of the embankment. Thickening the layers of an embankment on the outside somewhat so that the layers will be concave upward may also present sliding of the layers on each other. When the plans call for a very long and high embankment, it is sometimes best to construct first a trestle and operate the road over it. The trestle should have a life of at least five or six years, and during that time material can be brought from some excavation, perhaps several miles away, where it was perhaps loaded with a steam shovel, hauled by the train load, dumped with an "unloader," and allowed all the required time to settle, the whole being done for a cost per yard far less than it would have cost during the original construction. The method has the added advantage of permitting the road to be quickly opened for traffic and permitting it to quickly get on an earning basis, for such a trestle can be built more quickly than a very high embankment.

72. Classification of Earthwork. One of the most fruitful sources of legal contention between a contractor and his employer is the classification of excavated material when the work is paid for according to the classification of the material excavated. It is not only true that there is an insensible gradation from the softest of earth to the hardest of rock, but a material which is very hard when first exposed will sometimes crumble up after a very short exposure to the atmosphere. It is even true that some kinds of rock which are very soft when first taken out harden after exposure to the air, but this class of phenomena never has any influence on mere blasting for excavation. To avoid these disputes, some railroads require their contractors to satisfy themselves as to the character of the material to be excavated and then to make a single bid per yard which shall include whatever material is encountered. With all its advantages, this throws all the uncertainty on the contractor, and unless the competition is very great and the bidding close the contractor will usually add so much to cover that uncertainty that the railroad will pay more than it would on a classified basis.

The classification is usually made threefold—(1) solid rock, (2) loose rock, including shale and hard-pan, and (3) earth. Solid rock includes only such material as cannot be removed except by blasting, when it is found in masses exceeding one cubic yard. Loose rock includes boulders which are more than one cubic foot in volume and less than one cubic yard; also stratified rock occurring in layers of not more than six inches, when they are separated by strata of clay; also all material (not classified as earth) which *can* be loosened with a pick and bar and which "*can* be quarried without blasting although blasting may occasionally be resorted to." "Earth" includes all material not considered above—boulders not over one cubic foot in volume, all clay, sand, gravel, loam, decomposed rock and slate, and all materials which can be loosened for loading by a plow with two horses, or such as one picker can keep one shoveler busy. A brief consideration of the above classification, which is compiled from the best authorities available, shows the infinite opportunities for dispute as to classification.

TUNNELS — SURVEYING.

73. **Character of Surveying.** There are few kinds of surveying for engineering work where accuracy is of such high financial value and where it is so difficult to accomplish as it is in tunnel work. By the very nature of the case a tunnel is usually located in a region where it is very rough and all the surface surveys must be made on very steep slopes where accurate measurements are exceedingly difficult. The surveys in the tunnel itself are made in cramped quarters where light is artificial and the atmosphere is perhaps smoky. The difficulties will be elaborated as the methods for obviating them are discussed. Tunnels are generally excavated from each end. A very small error at either end will accumulate, especially if the tunnel is very long, until when the two headings meet there may be an offset which might actually necessitate a small reversed curve in the alignment. Therefore only the most refined measurements for distance, the

most refined leveling between the ends of the tunneling and the repeated measurements of all horizontal angles or the most precise prolongation of lines are to be used. All such work should be repeated and checked until the probable error of the work is so small that such error as may remain has no financial importance. The cost of such refined work is amply justified, because the lack of it may result in an error whose financial value might be very great.

74. **Surface Surveys.** The relative position of the two ends of the tunnel is first determined, *i.e.*, the azimuth and length of a line joining the two ends and the relative elevation. Usually a line is run on the surface which will be at every point exactly over the center line of the tunnel. When the tunnel is perfectly straight throughout, this is comparatively easy. Any curvature in the tunnel complicates the surveying greatly. A permanent station, from which a long sight can be run into the tunnel, is placed at each end. Then intermediate permanent stations are set so that adjacent stations are intervisible. These stations are first set approximately on line and then by repeated adjustments they are all set exactly on line. Any intermediate shaft can then be located from the adjacent stations.

Distance. The distance is sometimes determined, as in geodetic surveying, by triangulation and the measurement of a base line. Some of the great Alpine tunnels have been measured in this way. But for simpler work a direct measurement is made with a tape. Since the slopes are usually very steep, it becomes impracticable to hold any very great length of the tape truly horizontal. It is then also necessary to plumb down from the down-hill end of the tape to the ground. This is troublesome and also introduces an element of inaccuracy. And therefore "slope measurements" are often made, measuring the slope distance between carefully marked points and at the same time determining the difference of elevation. A simple geometrical calculation then determines the true horizontal distance. These marks may consist of needles set in wooden plugs supported on ordinary surveying tripods.

Levels. The above method includes the leveling. But if the ordinary method of leveling is used, especial care must be taken, since the slopes are very steep and the vertical distances to be overcome very great.

75. Underground Surveys. Station marks, corresponding to the stakes of ordinary surveys, cannot usually be placed in the bottom of the tunnel since they would be very quickly disturbed or covered over with debris. If the tunnel is timbered, the easiest method is to place the marks on the timbering, but this should not be done unless the timbering is very firmly in place and is not liable to be shifted. The better plan is to drill a hole in the roof of the tunnel, insert a wooden plug, and then set in the wood a small hook or nail which marks the exact point. Occasionally such marks are placed on the side of the tunnel. When placed in the roof there is the advantage that a plumb line, which must be illuminated by a lantern, may be swung from the hook or nail. A still better device is a plumb bob hung by a pair of cords attached to a "gimbel joint" on the bob. The bob has a little reservoir for oil and a wick exactly in the center which will furnish a flame which may be seen as far as necessary and which may be bisected by the cross hairs of the transit with great accuracy. Such "sights" can be reproduced whenever desired with great confidence that there is no appreciable variation. When a mere plumb line is used to sight at, it must be illuminated by some sort of lantern. This may be done by using a lantern with a ground glass which is placed behind the line, which is seen by its contrast against the illuminated background. If an ordinary lantern is used, it should be placed nearly in front of the line and pointing away from the transit, so that, without being seen from the transit, it illuminates the face of the line which then shows light against the darkness of the tunnel.

The leveling must, of course, be done with the level rod inverted so as to obtain the distance from the station point *down* to the line of sight. Of course this makes a corresponding difference in the calculations which must be carefully watched to avoid a blunder due to this change. This may be avoided by always placing a minus sign before any rod readings so taken, and then following the old rule of *algebraically* adding backsights and subtracting foresights and intermediate sights. Various devices are required to meet special conditions which test the inventive ingenuity of the engineer.

76. Surveying Down Shafts. In the case of very long tunnels it sometimes seems advisable to sink shafts at one or more points on the line of the tunnel, and when they have been sunk to the required depth, proceed to dig out the tunnel in each direction. For such work it becomes necessary to determine, at the bottom of the shaft, elevation, distance and alignment. If the shaft is vertical, as is usually but not always the case, the elevations are most readily carried down the shaft by means of a steel tape by methods which are obvious. Distance, which means in this case the longitudinal position in the alignment of the road of any given point, is readily transferred from the surface to the level of the tunnel by a very obvious application of the results of the next process to be described. Transferring the alignment with accuracy requires the utmost care and ingenuity. In principle it is very simple. Two heavy plumb bobs are hung on steel wires which are long enough to reach from the surface to the tunnel. At the surface they are placed on a line. Theoretically they should be on a line at the level of the tunnel. If a transit is so placed in the tunnel that its line of collimation passes simultaneously through both wires, it is in the line of the tunnel. Such is the simple outline; some of the difficulties are as follows:

Although the wires are set as far apart as possible along the line of the tunnel, the distance is absolutely limited by the size of the shaft. Any minute error in the location of these lines (say eight feet apart) will be greatly magnified when the headings are run out 6,000 or 7,000 feet in each direction from the base of the shaft, as was done in the case of the Hoosac tunnel. The currents of air up a tunnel shaft have considerable effect in swaying the wire from a true vertical. In the case of the Tamarack shaft, 4,250 feet deep, the wires were 0.11 foot farther apart at the bottom than at the top. The discrepancy *in that direction* had no effect on the alignment, but if the wires had an error whose combined effect in that direction was 0.11 foot, the lateral error while unknown was perhaps as much or more. The uncertainty was therefore in that case very great. Incasing such wires for the whole distance in a box reduces the effect of air currents. The plumb bobs are swung in pails of water or oil at the bottom and their locations noted as carefully as possible, taking the mean position of the vibrations which cannot be altogether overcome. Marks are then set at the bottom of the shaft (but in the roof of the tunnel) from which plumb lines can be hung. A transit can then be set by trial so that its line of collimation simultaneously passes through both wires.

TUNNEL DESIGN.

77. Cross-Sections. The variety in the cross-sections which have been adopted is due to the fact that there are no absolute requirements which determine the design except in a general way. If the tunnel passes through such very soft soil that there is excessive pressure the form should be circular or nearly so. While the size of the rolling stock is in one sense a limitation, yet the clearance should be considerable, partly for the reason of allowing something for a possible settlement of the lining. A majority of the sections used have a semi-circle or semi-ellipse surmounting a rectangle or trapezoid. Even when the ground is so soft that lining is required not only at the top but also at the sides and bottom, the same general shape will be used except that the straight lines will be replaced by arcs of circles concave to the center of the tunnel. Illustrations of cross-sections will be shown under a subsequent section.

A tunnel almost invariably strikes one or more veins of water which immediately begin to discharge into the tunnel, which thereafter becomes the drainage outlet for such water. This necessitates an adequate provision for drainage. In a double track tunnel the drain will usually be placed between the two tracks, but with single-track tunnels they must necessarily be placed on each side. Figs. 48 and 53 will illustrate this feature.

78. Grade. Many tunnels are situated at the summit of two grades, which are very probably the ruling grade of the road. In such a case it is possible to make the ends of the tunnel at practically the same level and have no grade in the tunnel except a slight summit near the middle of the tunnel which shall furnish grade for drainage. Grade for drainage should never be omitted--about 0.2 per cent grade is required. But tunnels are frequently necessary as parts of a grade which is very possibly the ruling grade of the line. In such cases the grade should be very materially reduced while running through the tunnel. The atmospheric resistance in a tunnel is greater, the rails are apt to be wet and slippery and the tractive power therefore less, while the consumption of the limited supply of oxygen by the locomotive and the poisonous fumes cast off, especially when the engine is working hard, is a source of actual danger to the engine crew and even to the passengers. Therefore a generous reduction of grade should be made, although the precise amount of compensation required is hardly computable.

79. Lining. The lining required varies from no lining, such as may be permitted when the rock is so firm that it will be absolutely self sustaining and will not disintegrate upon exposure to the weather, and a lining of the very heaviest and strongest cutstone masonry which should be used when the ground is subject to extensive settling. This condition is far worse than any mere fluid pressure. Many American tunnels have been constructed with a permanent lining of timber, such as is illustrated in Fig. 47. In other cases the cross-section of a tunnel has purposely been made somewhat larger than necessary, so that when the timber lining required renewal a masonry lining could be built inside of the timber lining without encroaching on the required clear cross-section and without requiring any disturbance of the timber

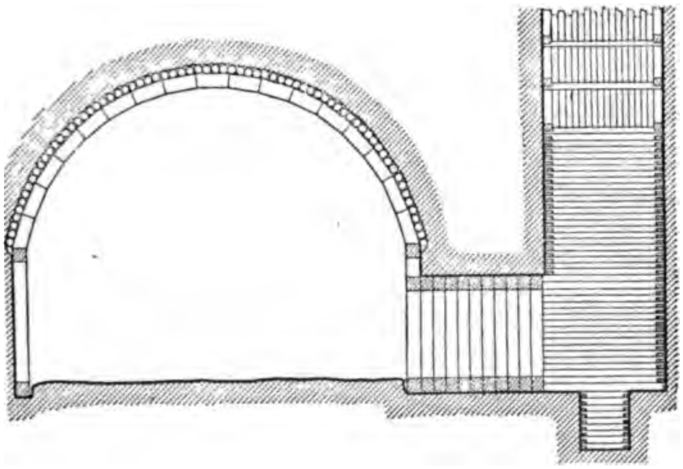


Fig. 47. Connection with Shaft, Church Hill Tunnel.

lining. In this way the heavy expense of the masonry lining could be deferred until a time when the road would probably be better able to pay for it. True economy requires the best of cement masonry. When, on account of an unintentional fall of rock outside of the nominal excavation lines a space would be left between the lining and the line of the excavation, such space should be filled with broken stone well packed in or even with concrete or solid masonry.

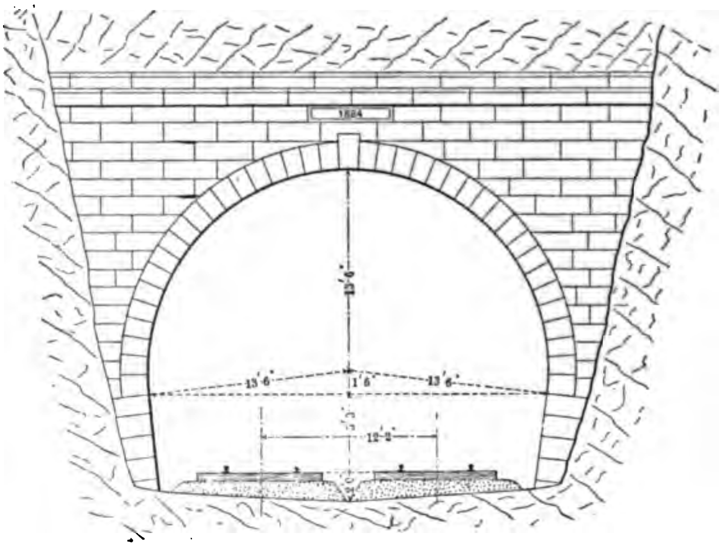
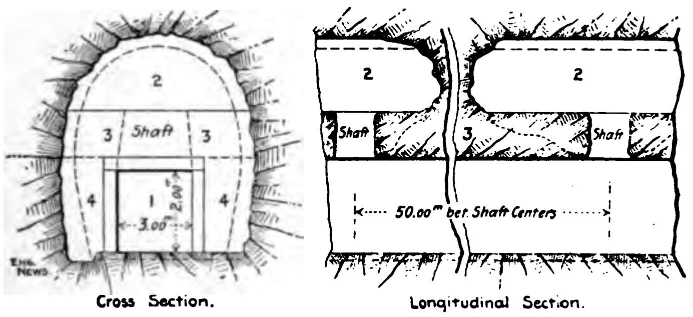


Fig. 48.

80. Portals. Although no calculations can be made to determine the forces acting on a portal, it is readily seen that they are sometimes very great, as they must often prevent a tendency of the face of the mountain to slide down over the tunnel outlet. In Fig. 48 is shown a typical portal. These are sometimes made very elaborate architecturally, but the leading feature of the design must be its massiveness. It must act as a retaining wall against the direct action of the slope. If there is also a considerable stretch of open cut at the entrance to the tunnel, then the design is really simplified by walls on the sides of the cut which will act as buttress walls to the portal. Some of the most difficult construction of a tunnel may occur at the portals. It is here that the thickness of the natural "roof" of the tunnel runs out to practically zero. Considerable thickness is required before it will become self-sustaining enough to give opportunity to place the timbering. The surface soil may also be so loose that the excavation below starts a landslide. Therefore a very heavy timber frame must be constructed outside of the line of the proposed portal and must be very heavily braced to withstand a probable tendency to a landslide. In one case a shaft was sunk a short distance from the portal; tunnel excavation and permanent masonry lining was at once commenced, running back toward the portal. As the surface was approached the thin roof was so thoroughly supported that no serious difficulty was encountered from a landslide.

TUNNEL CONSTRUCTION.

81. General Principles. A large majority of tunnels require a lining because the material through which they are excavated cannot be depended on to be self-sustaining. Except in sub-aqueous work, all material is self-sustaining over a small area and for a short time, a time long enough so that after a small area has been exposed a support even though temporary may be placed which will prevent a fall at that place. Since there are all gradations in material from the hardest of rock to the softest of quicksand, there are likewise gradations in the methods to be adopted, in the promptness with which timbering must be placed to support exposed areas and also in the extent of area which may be safely exposed before timbering is placed. All methods agree in excavating one or more headings in advance of the full sectional excavation.



Cross Section. Longitudinal Section.

Fig. 49.

These headings are sometimes made at the top, sometimes at the bottom and sometimes at the two lower corners. One good effect of such headings is to drain the soil in advance of the main excavation and thus facilitate the subsequent work. These headings are then enlarged until at last the full sectional area, including that required for the lining, is obtained. The construction of the lining follows closely, so that in a stretch of perhaps less than 80 feet may be seen all stages of the work, from the initial heading to the finished tunnel completely lined.

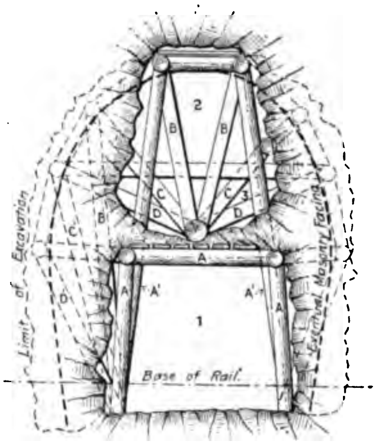


Fig. 50.

82. Methods. The limitations of this paper will not permit a complete discussion and description of the various methods which are used in this work. Some illustrations are shown to give the student such a grasp of the general principles involved that a generous application of common sense may enable him to accomplish some of the plainer and simpler problems. The timbering must be so designed and placed that there will be little or no tendency for the pieces to slip on each other and that any added pressure will only bind the framing still tighter together. The timbering should never fail except by absolute crushing, and its cross-section should be made such that it may withstand any probable pressure. An inspection of the illustrations will illustrate this.

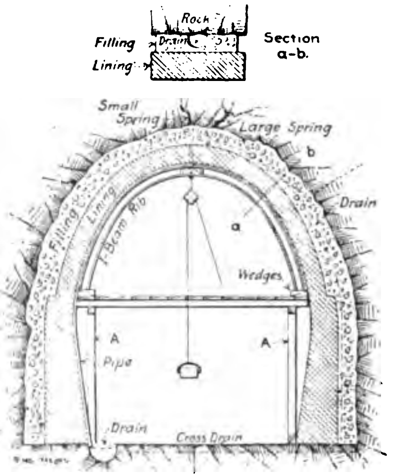


Fig. 51.

Much of the difficulty of tunnel work arises from the limited space in which the work must be done. A well-devised system of removing excavated material as rapidly as it is loosened and of handling the materials for the lining and placing them in position is therefore an absolute necessity. The use of small cars on rails is usually advisable. With a tunnel of any considerable length, artificial ventilation during construction is necessary, especially if blasting is required. As before mentioned, compressed air may be used to operate the drills for blasting and this may supply the need. But where no blasting is required, and sometimes even when compressed air is used, ventilation by fans is necessary. The fans and engines for operating them are of course placed outside the tunnel and the fresh air is discharged from a pipe where desired.

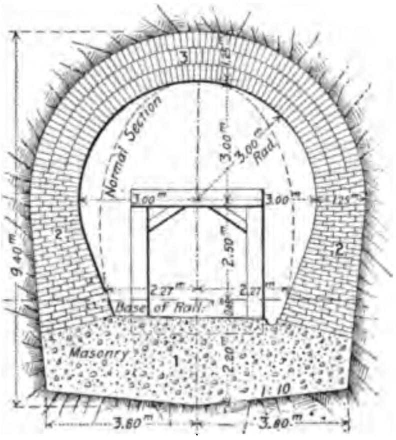


Fig. 52.

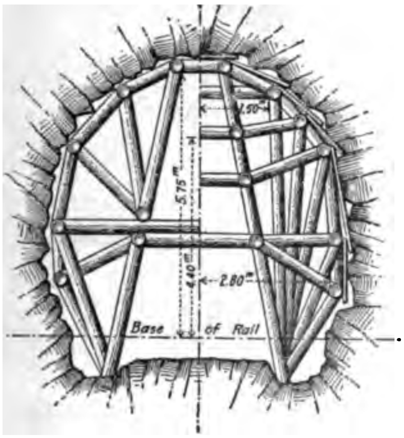


Fig. 53. Fig. 54.

TRESTLES.

A trestle consists of two essential parts, the sub-structure framework and the floor system. Since the floor system is essentially independent of the sub-structure, it will be separately described. There are two systems of building the sub-structure, by piling and by timber frames.

83. Pile Trestles. These are limited in height to the length of a single pile which may safely be used. The length of pile required must include the necessary depth to which they must be driven. On this account 30 feet above the ground is about the limit of height of a pile trestle. With exceptionally long piles higher pile trestles might be built, but framed bents would be preferable. Usually four piles are considered sufficient for single track, although more are sometimes used. The inner piles are always made vertical but the outer piles are sometimes battered so as to give the trestle a greater resistance against a lateral thrust.

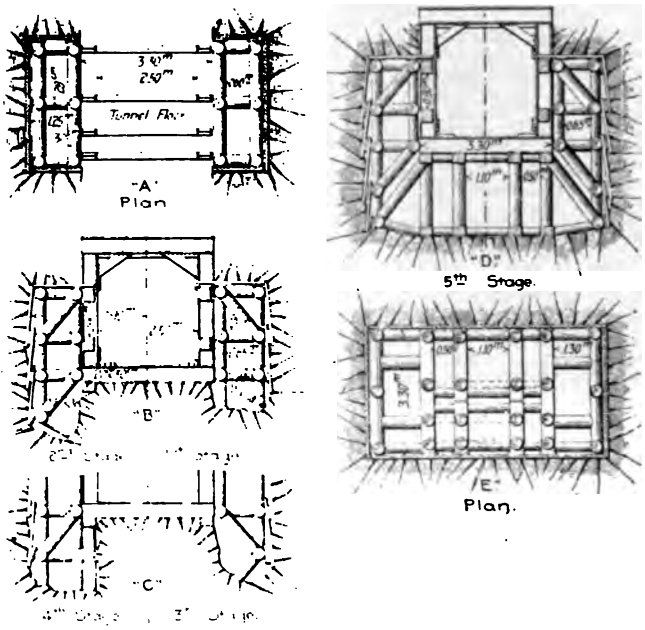


Fig. 55.

For a high trestle (greater than 10 feet) this thrust is best taken up by sway bracing. The piles are surmounted by a cap which is generally 10 × 12 in., or perhaps 12 × 12 in. A still better form of cap is the "split cap," which consists of two pieces bolted together as shown in the detail of Fig. 60. Other methods of joining the cap to the piles are illustrated herewith. The construction using drift bolts is perhaps the cheapest and most quickly erected, but it has the disadvantage that repairs are difficult, and if the trestle is merely temporary it is almost impossible to remove it without ruining the timber for future use. The mortise and tenon joint is perhaps the most common for good practice. The piles should be not less than 14 in. in diameter at the butt and 7 in. at the top, exclusive of bark, which should be removed before driving. The soft durable woods such as cedar, cypress, pine and redwood are best for piles that are not driven in a stream where they may be subject to the blows of floating ice. The oaks are stronger but are less durable in the ground. The caps are preferably made of hard wood such as oak or yellow pine. They should be about 14 ft. long for single track. The sway braces are generally 3 × 12 in. and are usually spiked with ⅜-in. spikes 8 in. long.

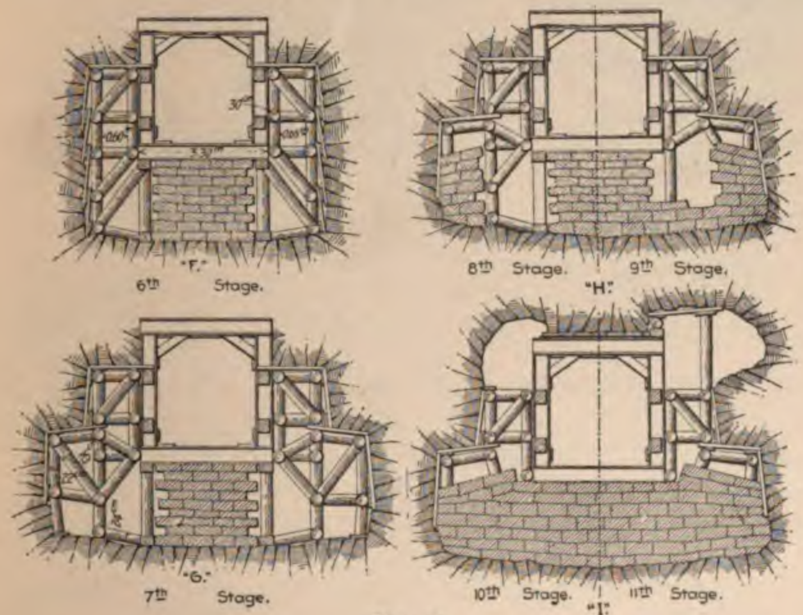


Fig. 56.

84. Pile Driving. Piles are usually driven by means of a hammer weighing 2,000 to 3,000 pounds, which is raised between guides to a height of perhaps 25 feet and allowed to drop onto the head of the pile which is suitably set between the guides. A very cheap way is to raise the hammer by horse power, and then loosen a clutch which allows the hammer to fall freely. A still better way is to use a portable engine which winds the hoisting rope around a drum. Sometimes the falling hammer is required to draw the rope and unwind the drum as it falls. On the one hand this obviates the use of a clutch and even permits more rapid blows, but on the other hand the force of each blow is very materially weakened and the method may be used by a dishonest contractor to falsely indicate a high resisting power of the pile. Excessive driving has been known to fracture a pile underground and render it almost useless. The action of the hammer splinters the top of the pile, causing it to "broom." This action very greatly reduces the effectiveness of the driving. This is largely prevented by chamfering off the top of the pile and driving on a wrought-iron ring, which has a section of about $\frac{1}{2} \times 2$ in. and of a suitable diameter. The frequent removal of all crushed wood from the head of the pile by means of an adze is amply justifiable in spite of the delay caused. Piles should be driven until their resistance as indicated by the penetration for a single blow is as great as is required. The most commonly used formula is that known as the "Engineering News" formula, which when used for ordinary hammer driving is as follows:

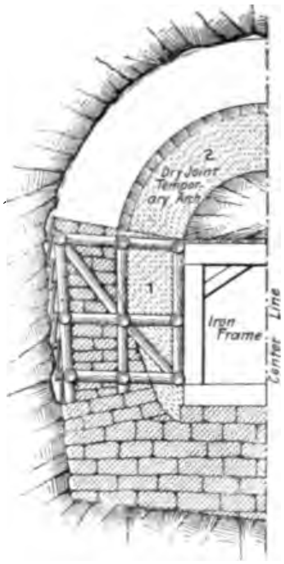


Fig. 57.

$$R = \frac{2wh}{s+1} \tag{52}$$

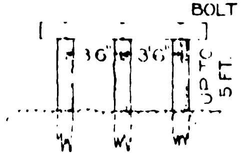


Fig. 58.

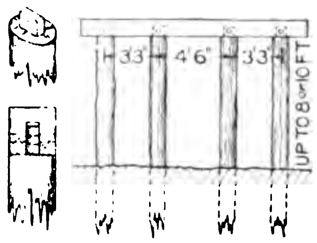


Fig. 59.

In this formula R is the *safe* load on the pile, *w* is the weight of the hammer, both in pounds, *h* is the height of the fall in *feet*, and

s is the penetration in inches of the pile during the last blow. Sometimes the average penetration during the last five blows will give a more reliable value.

Example 1. A pile was driven with a 2,500-lb. hammer until the total penetration during the last five blows was 13 inches. During those blows the hammer dropped 23 feet. How much is the safe load?

$$\frac{2\,wh}{s+1} = \frac{2 \times 2{,}500 \times 23}{(\frac{1}{5} \times 13) + 1} = \frac{115{,}000}{3.6} = 31{,}944 \text{ pounds.}$$

Example 2. It is required to drive piles with the above hammer until the indicated resistance is 25,000 lb. What should be the average penetration during the last five blows, the fall being then 22 feet?

$$25{,}000 = \frac{2\,wh}{s+1} = \frac{2 \times 2{,}500 \times 22}{s+1} = \frac{110{,}000}{s+1}$$

$$s = \frac{110{,}000}{25{,}000} - 1 = 3.4 \text{ inches.}$$

Another form of pile driver is that known as the "steam pile driver." This consists essentially of a hammer which is directly attached to a piston in a steam cylinder. The hammer, weighing about 5,500 pounds, is raised the height of the cylinder, which is about 40 inches, and then falls freely. Although the fall is so much less the blows are very rapid—about 75 to 80 per minute. The practical effect of this is that the soil does not have time to settle between the blows and the penetration is more easily accomplished, while the ultimate resistance is as great as before. On this account the constant "1" in the denominator in equation 52 is changed to 0.1 and the formula then becomes

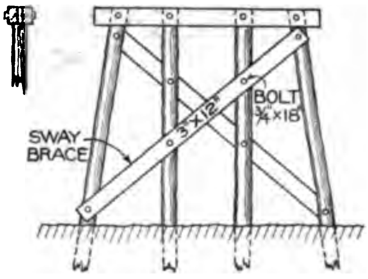


Fig. 60.

$$R = \frac{2\,wh}{S + .1} \qquad (53)$$

FRAMED TRESTLES.

85. General Form. Although there are multitudinous variations in the details of construction, a very large proportion of framed trestles are constructed substantially in accordance with the typical design shown in Fig. 61. The outer posts are generally battered about 1:6; the cap and sill are mortised to the posts, although split sills and caps can be used advantageously. The sway bracing should be bolted on. Although the mortise and tenon joint are most commonly used, there are many other designs. The "plaster joint" is one of the most common. This consists of two pieces of 3-in. plank which are placed on each side of the joint as in Fig. 62, and are bolted through and through. This form has the merits of cheapness and facility for taking out and renewing any decayed or injured piece. Iron plates are sometimes used in a similar manner. Dowels and drift bolts are also used, but as mentioned before have many objections.

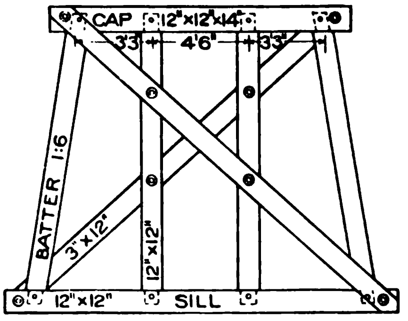


Fig. 61.

86. Multiple Story Construction. A single-story framed trestle should not be made over 25 or 30 feet high. Additional height is obtained by dividing the height into two or more stories. Then since all the upper stories must be of uniform height the odd amount must go to the lower story, as is illustrated in Figs. 63 and 64. Some plans have these stories absolutely independent of each other. This simplifies the construction and makes repairs easy, but the trestle will be lacking in stiffness. These illustrations should be studied with special reference to the design of the lateral bracing of the individual bents and also the longitudinal bracing of the trestle as

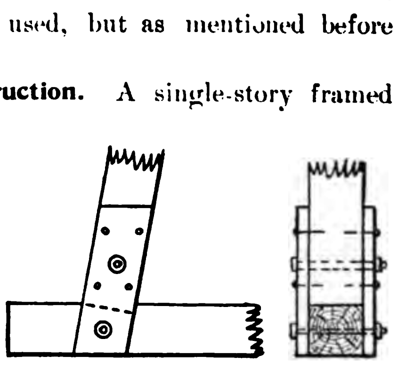
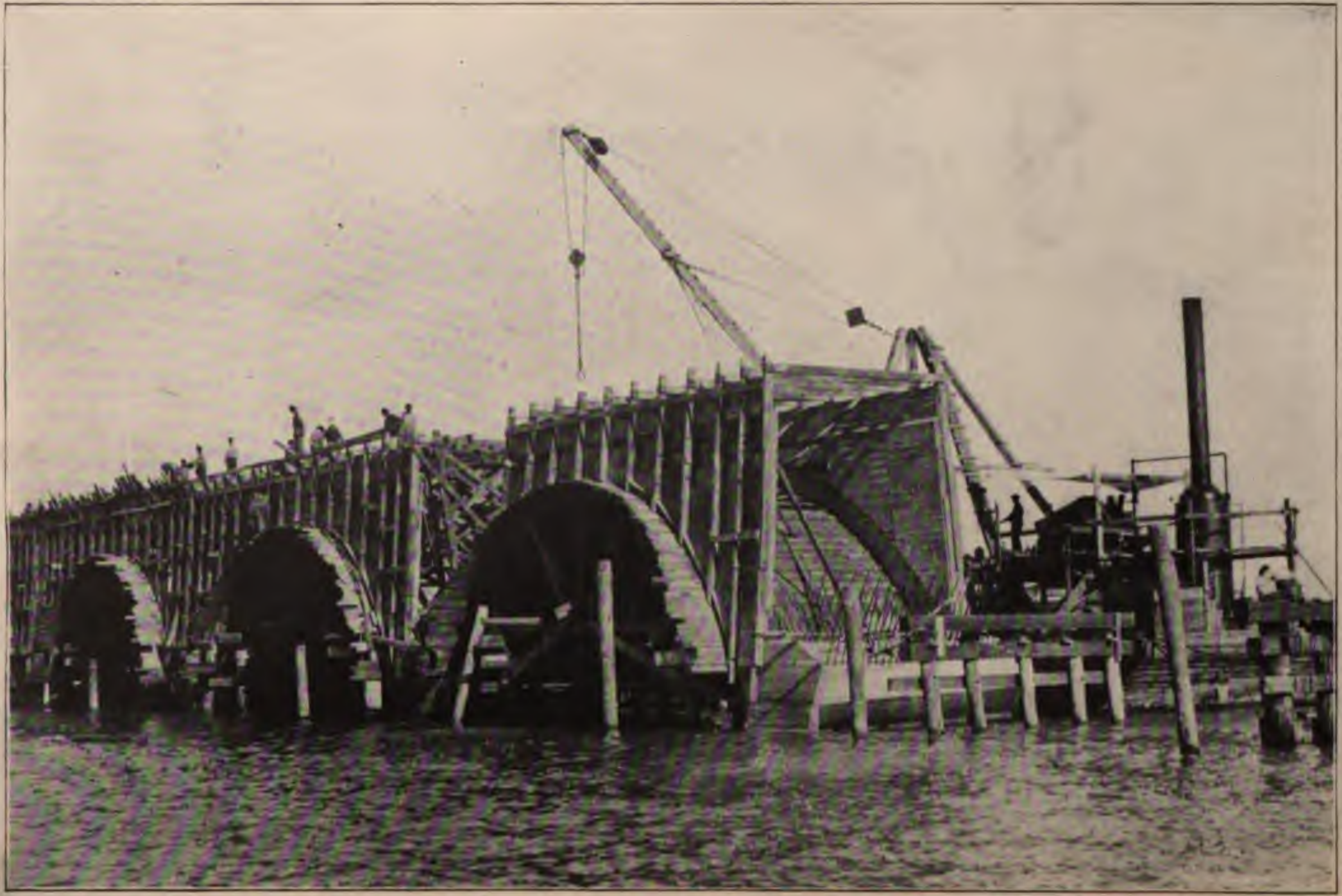


Fig. 62.



PLACING OF ARCH FORMS IN CONSTRUCTION OF REINFORCED-CONCRETE RAILROAD VIADUCT

Long Key Viaduct on Key West extension of Florida East Coast Railway. The reinforcing rods protruding from the pier are continued over the ring of the arch.

a whole. Note that the lateral bracing always runs to some point where two or more pieces intersect, and when possible it is so designed that even the intermediate points are a common point for several pieces. A thorough bolting at these points greatly stiffens the structure. The span between the bents varies from 10 feet to 18 feet. For high trestles economy requires that the number of bents shall be reduced as much as possible, which means that the spans should be increased. But this increases the requirements for the floor system, and also the load to be carried by each bent. 18 feet is about the safe limit for railroad rolling stock on untrussed wooden floor beams.

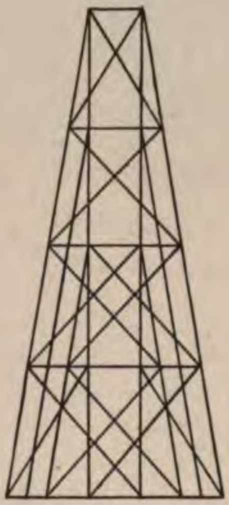


Fig. 63.

87. Foundations. Trestles are frequently to be classed as "temporary" structures. Such will justify the use of a foundation of a more temporary character than could be tolerated for permanent work. When time is important and the ground soft, piles are sometimes driven and sawed off a little above the ground. They are so placed that a pile comes as nearly as possible under each post of the trestle. Of course such foundations must be considered as very temporary in character, as they will speedily decay to such an extent as to render them unsafe. Locust or chestnut are preferable for this purpose.

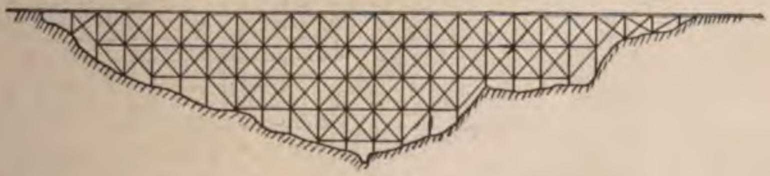


Fig. 64.

Another form which is even easier to construct, but which is, if possible, still more subject to decay, is the "mud-sill" foundation. The sill of the trestle is set on a number of timbers placed transverse to the sill, the timbers being about 12 × 12 in. × 6 ft.

If the ground is very soft even these timbers may be set on two or more long timbers, laid parallel to the sill, as shown by the dotted lines in Fig. 66.

When the trestle is intended as a permanent structure, and especially when it is intended to ultimately replace it with a steel viaduct, a stone foundation may be used. If built of rough rubble the wall should be about 4 feet thick. If the masonry was of a better class the wall need not be so thick, but the cost would be about the same. Usually a single continuous wall would be built, but if the trestle is very high the usual batter adopted for the side posts will make the sill very long. With some designs of trestles, depending, however, on the plan of the posts, it is permissible to save some masonry by omitting portions of the wall between the center and the ends.

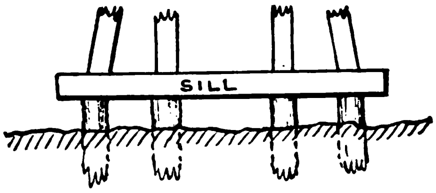


Fig. 65.

88. Abutments. At each end of a trestle the natural surface usually approaches the grade line by a slope. If stone foundations were built for the bents, then stone abutments would be built which would act as a retaining wall for the last few feet of rise and which would support the last stringers. When piles are used an abutment such as indicated in Fig. 67 is used. When no piling is used, an abutment may be made in the form of crib work. Sometimes one end of the last line of stringers is merely buried in the earth or is supported on a "mud sill." Of course, all of these latter methods should be considered as temporary. The danger in them lies in the chance of these places being neglected and the decay unnoticed until the decayed timber suddenly gives way and a costly accident is the result.

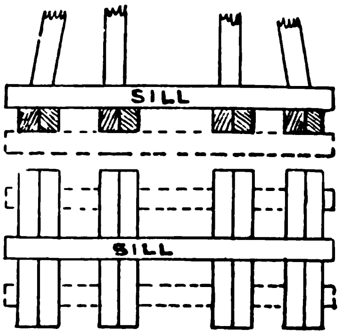


Fig. 66.

TRESTLE FLOOR SYSTEMS.

89. Stringers. The design of stringers depends somewhat on the cost and practicability of obtaining timbers of the length and thickness that theory would call for as the most economical size. Sound timber of the required length, and more than 16 or 17 inches in thickness, is scarce and correspondingly costly. The required transverse strength for stringers is, therefore, obtained by taking as large pieces as may be readily obtained, setting them on edge or with the largest cross-sectional dimension vertical, and then bolting two or more of them together side by side. Two timbers, each 8 × 16 in., bolted together side by side with the 16 in. dimension vertical, are practically as strong as a 16 × 16 in. timber, and are very much easier to obtain in a sound condition. These stringers should preferably extend over two spans, the lines "breaking joints." This requires pieces from 20 to 32 ft. in length. The pieces of each line should be separated by "separators," which are sometimes cast-iron spools, 1 or 2 in. long, which are strung on the bolts, or are sometimes made of pieces of plank about 6 feet long. Bolts are run through the stringers and separators. The plank separators thus serve to tie the consecutive stringers together. The chief object of the separators is to permit air to circulate freely around the timbers. Placing the rough sawed timbers side by side would allow water to soak in and be retained, so that decay would be very rapid. The design of stringers is susceptible of exact calculation for the transverse strength required, but as this is a direct application of the subject of "Strength of Materials," the method of design will not be elaborated here, except to call attention to the fact that the stringers must be designed to withstand not only transverse strains, but also shearing and crushing across the grain where the stringer rests on the cap. A very high and narrow stringer might

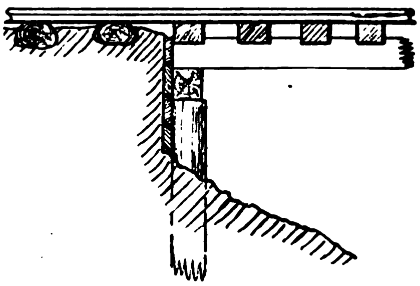


Fig. 67.

have sufficient transverse strength, but might be so narrow that it would fail by shearing along the neutral axis. The same stringer might also have such a small area where it rests on the cap that the safe limit of crushing across the grain might be exceeded. The safe values to be used with various kinds of wood for these various stresses may be found in many handbooks. The following dimensions have the approval of very extensive practice:

Clear span.	Number of pieces under each rail.	Width.	Depth.
10 feet.	2	8 inches.	15 inches.
12 "	2	8 "	16 "
14 "	2	10 "	17 "
16 "	3	8 "	17 "

90. Corbels. A corbel in a trestle is the name applied to a timber placed on the cap of the trestle bent and on which the stringers rest. The argument in favor of their use seems to be that they greatly increase the area of pressure on the seat of the stringer. They can also be utilized to bind together two abutting stringers. But although the crushing of the end of the stringer may be prevented, the area of contact between the corbel and the cap must be considered, to determine whether crushing might occur there. There is great diversity of opinion regarding their use. Many standard designs do not use them.

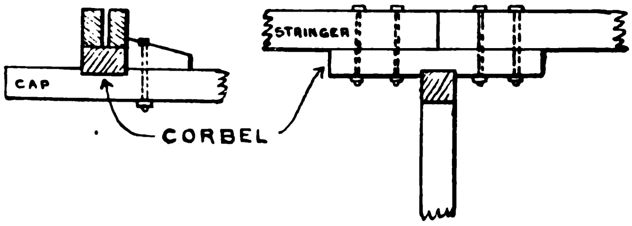


Fig. 68.

91. Guard Rails. These are timbers varying in size from 5 × 8 in. to 8 × 8 in. which are placed near the ends of the ties. They are usually notched about 1 inch at each tie so that they really form tie spacers and thus prevent the ties from becoming displaced if a car becomes derailed on the trestle. They should be bolted to the ties at every third or fourth tie. There are various methods of jointing the ends of abutting pieces. The method shown in the figure is perhaps as good as any.

The name guard rail is also applied to the inner guards which are placed about 10 in. inside of each rail. These are usually the ordinary T-rails used for traction. These rails are relied on to keep the cars on the trestle if they should become derailed. If a car should become so displaced that the wheel reached the outer wooden guard rail, it would probably catch on it, slew around and jump over. Therefore the sole function of the outer wooden guard rail is to keep the ties spaced and in place.

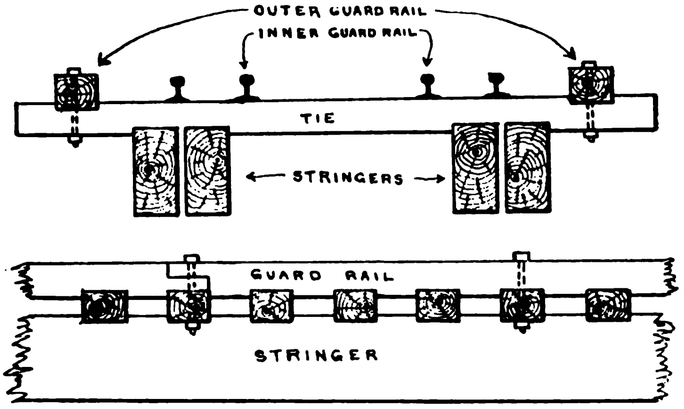


Fig. 69.

92. Trestle Ties. Trestle ties are always made of sawed timber. They are longer than ordinary ties—usually 9 to 12 feet. The depth is frequently much greater, with the apparent idea that they may act as a flooring that will support the rolling stock if it should become derailed. For a similar reason the spacing is made very much closer—generally equal to or less than the width of the ties. Sometimes even the ties are notched on the underside where they rest on the stringers. Some plans have a stringer run under each guard rail. Then bolts will be run through the stringer, tie and guard rail at every third or fourth tie. If the ties have been notched down on the stringers and the guard rail is notched down on the ties, then these bolts will tie the whole system immovably together.

93. Super-Elevation of the Outer Rail on Curves. Locating a curve where a trestle is also necessary, is in general very objectionable, but it is sometimes unavoidable. The objection lies not only in the fact that a very considerable force is required to guide the train in its circular path, but the force is variable, depending on the variable speed, and there are apt to be oscillations of unknown force which will still further rack the trestle. Nevertheless these forces must be provided for as closely as possible. If all trains moved along the trestle at precisely the same speed, the problem would be comparatively simple. The whole floor system could be designed to resist the thrust due to the forces developed at that speed. But since the speed may vary down to zero, and the train start from rest while on the trestle, which of itself will introduce new strains, the construction which is best for the highest speed is not the best when the train is standing on the trestle, or when it is starting, and *vice versa.* A few of the many designs which have been used will be illustrated, together with a brief comment on the advantages and disadvantages of each design. The required super-elevation of the outer rail and the method of computing it will be discussed in the chapter on track work and track laying.

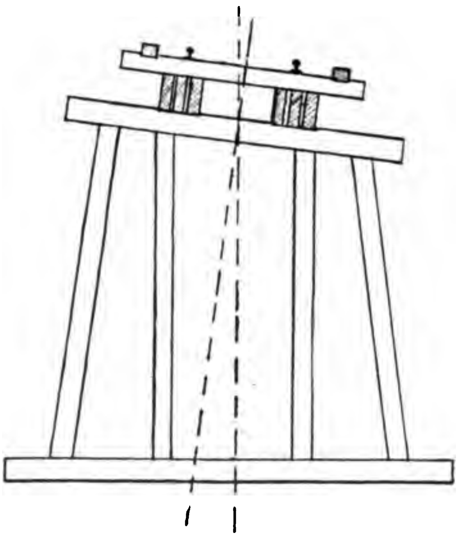


Fig. 70a.

(*a*) *Inclining floor system and cap; sill horizontal; outer posts longer.* The construction of the trestle bents is more complicated, but that of the floor system is simplified. Since the stringers do not stand vertically there is a tendency for them to twist when the train is stationary.

(*b*) *Placing wedges under the ties at each tie.* Two or more wedges are required for each tie. Each wedge is bolted by two bolts. The number of pieces is very great, but there is the advantage that the ties are not notched or weakened in any way. If for any reason a different super-elevation is desired the wedges are all that need be changed.

(c) *Placing a wedge under the outer rail at each tie.* This is similar to the last method, but requires fewer pieces. If the super-elevation is slight, either very long spikes must be used or lag screws may be used which will run through the wedges into the ties. For a greater super-elevation the wedge must be fastened, as shown in the illustration.

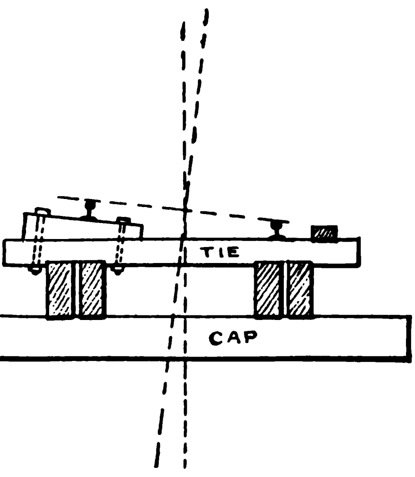


Fig. 70c

(d) *Corbels of varying height.* The whole floor system is tipped as in *a*, but the trestle bent is as usual, with cap and sill horizontal. In all such cases, where the axis of the post is vertical, the lateral bracing of the bent should be made extra heavy. It should be especially noted whether the center of pressure under extreme conditions reaches the sill too near the outer end of the sill.

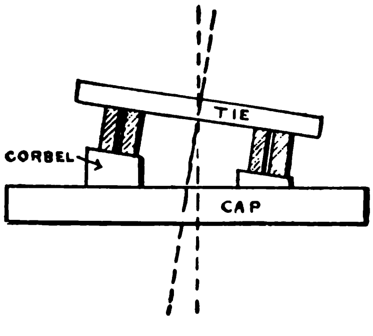


Fig. 70d.

(e) *Tipping the whole bent on its foundation.* The advantages and disadvantages of the method under some conditions are obvious.

(f) *Notching the cap.* The method is mentioned on account of its frequent use, but the disadvantages are very great. The cap is weakened by the notching. Either the stringer or the tie must be notched at each tie. Usually the tie would be hopelessly weakened if it were notched sufficiently, and, therefore, the stringers must be notched. The method is costly in construction and objectionable when made. The above methods are types of a great variety of plans of construction which have been suggested and tried.

94. Protection Against Fire. One of the strongest objections against the use of trestles is the danger from fire. Sparks from the locomotive or wayside fires kindled by tramps and others may destroy them, or, what is still more dangerous, may slowly eat into the timbers until they are weakened beyond the danger line, and yet, because the effect of the fire is not apparent to a careless inspection, it may result in an appalling accident. The danger from falling coals from the locomotive firebox is largely obviated by constructing a solid floor or trough on the stringers, the trough being filled with ballast and the ties set in the ballast as usual. Another method is to cover the stringers and caps with sheet metal. A very long trestle generally deserves the protection of a special watchman or track walker. Means for fighting a fire when discovered are provided by reservoirs of water, made perhaps from halves of oil barrels, which are placed on the trestle at intervals of 300 feet. Three or four ties are made about 4 feet longer than the usual length. These form the floor of a platform, which, when provided with a railing, forms not only a place for the barrel, but also a refuge bay for the track walker, who may be on the trestle when a train is passing.

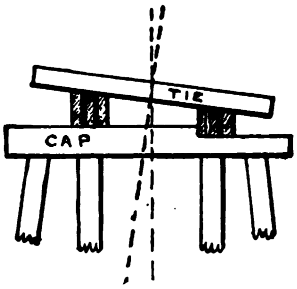


Fig. 70*f*.

95. Choice of Timber. When a railroad is being run through a virgin country where timber is plentiful and there is frequent occasion for trestles, it pays to take a portable sawmill to the spot and saw the timber as required. Under such conditions any one of the various kinds of timber which are ever used for building purposes will answer. If necessary, the cross-sections can be increased to correspond with the reduced strength of

a weaker wood. But when the wood must be transported a considerable distance and it is practicable to choose among various kinds of wood, the selection should be made according to favorable qualities. Ties and guard rails should, if possible, be of oak. Stringers should be made of oak or pine. Since one of the chief uses of corbels is to relieve a dangerous pressure across the grain they should be made of the hardest wood obtainable, such as oak, hickory or ash. The bents of a framed trestle may be made of almost anything, but oak, pine or fir are preferable when obtainable. If the sills are liable to become buried somewhat in the ground so that rain will not readily be shed, then some wood like cedar, which is very long-lived under ground, might be preferable, but the strength as posts will be somewhat less than that of oak. The chemical treatment of timber for trestles is seldom used, except for trestles which are partly immersed in water where the *teredo navalis* is found. Trestles are usually considered to be so cheap and temporary that conditions which would justify the additional expense of chemical treatment would also justify the immediate construction of a permanent structure of steel or stone.

On the accompanying folding plate is shown the standard plans for a framed trestle as adopted by the Great Northern Railroad. Many of the details shown will verify those already mentioned, while in other cases the variations in detail represent practice equally good. The plate is well worthy of a long and close study.

CULVERTS.

96. Pipe Culverts. The scarcity of stone suitable for making a "box" or "arch" culvert has led to the adoption for many localities of pipe culverts, the pipes being made of tile or iron. Pipes have several very great advantages. Their form is hydraulically better than any rectangular form and the surface is usually very much better than an ordinary masonry culvert. Therefore they will discharge a far greater volume of water than a box culvert of equal area. They are very easily placed without skilled labor. Sometimes they are set inside of a wooden box culvert temporarily placed during the construction of the road. When one pipe of the size which it is desired to use has insufficient area two or more pipes may be used side by side. This feature is of special value when the head room between the bed of the stream and the grade line is limited. Iron pipe usually has such inherent strength that there is little need for special care in securing a foundation for the pipe. A little block of concrete at each joint is sufficient for ordinary cases, but tile pipe requires a secure foundation.

The danger to the pipe does not lie so much in the mere static pressure of the earthwork embankment above it as in the effect of settlement of a "green" embankment. If the pipe is

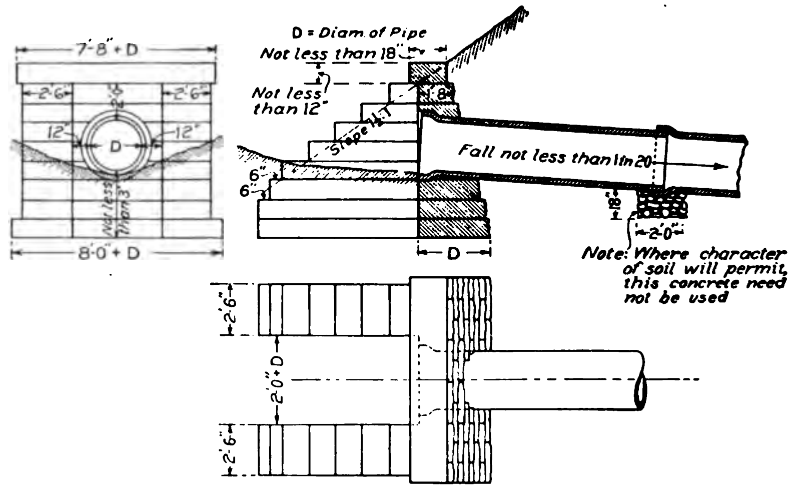
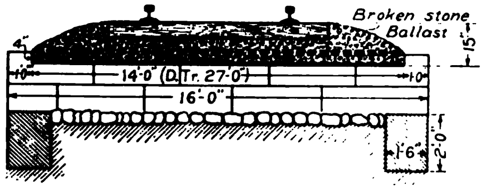


Fig. 72. Pipe Culvert.

laid on the natural soil, which might be tolerated if it is very firm, a bed should be carefully scooped out so as to fit the pipe as closely as possible. A better plan is to place a thick layer of broken stone or brickbats and ram them to the proper form as a bed for the pipe. A still better plan is to place a layer of concrete under the whole length of the pipe. The required slope of the pipe depends somewhat on the accuracy of the laying and on the permanency of the work. A slope of 1 per cent is ample, provided the grade be made and maintained uniform, but the effect of settlement may be to change such a grade to a negative grade, which would prevent the water from being carried off. Some standard plans therefore require a grade as steep as 1 in 20. At each end of the pipe there should be a substantial head wall of masonry. Some standard plans make this wall very large and heavy with elaborate wing walls. These are justifiable on the grounds of preventing the water at the upper end from scouring around the ends of the head wall or of preventing the outflowing water from scouring away the bed of the stream and thus undermining the lower head wall.

An iron pipe can be used if necessary very close to the ties, but a tile pipe should have a cushion of at least three feet between the tile and the ties. The joints in the pipe should always be caulked. Clay puddle is much used for this purpose and when it is of good quality and the work well done, the results are satisfactory, but if clay puddle cannot be obtained it is better to use hydraulic cement. The cost of the cement is an insignificant item considering the value of the result.



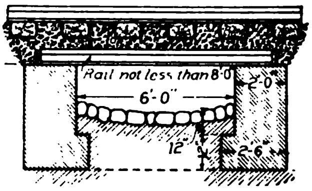
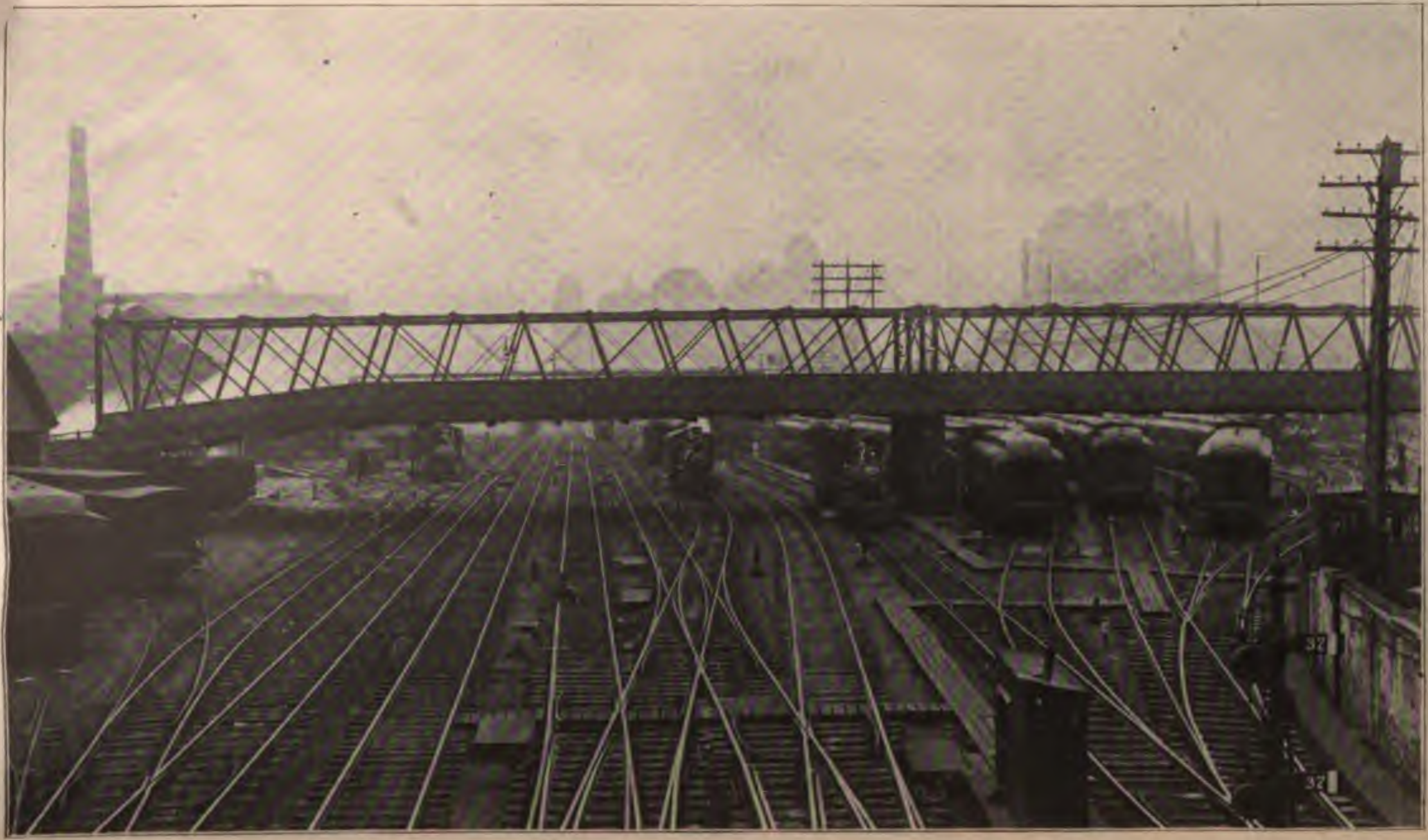


Fig. 73. Old Rail Culverts. Fig. 74.

97. Old-Rail Culverts. These have an especial value when the head room between the bed of the stream and the rails is small, and when it is also necessary to provide for a considerable flow of water. The old rails, even when worn out as rails, still have a considerable strength as girders and a continuous layer of them is amply strong enough to carry the roadbed and the traffic over a six-foot opening. The rails may be bound together by means of tie rods run through the webs of the rails, but they may also be confined by stones at each end of the seat course on each abutment.

Another advantage of this form of opening, over the common plan of supporting the ties on stringers or steel girders, is that in this plan the ballasted roadbed is continuous. This is a great advantage both from the standpoint of smooth riding and of safety.

98. Cattle Passes. When an embankment crosses a farm, cutting it in two, it becomes necessary for the road to provide a passage way through the embankment for the use of cattle and farm wagons. The cost of such a structure is compensated by the relief of the company from damages due to the cattle crossing the road at grade. These openings are sometimes built as large stone arch culverts or as old-rail culverts, especially if there is liable to be a storm-water flow through them. Another method is to set two trestle bents at the requisite distance apart; 3-inch planks are set behind the bents to hold the earthwork embankment; the stringers are notched down so as to take up the thrust of the embankment. This method naturally applies to embankments which are from about 8 to 15 feet in height. The disadvantage incident to all wooden structures set in earth also applies here. There is also the disadvantage of a break in the continuity of the ballasted roadbed, as well as the danger due to an accident from fire destroying or weakening the structure. When the head room is limited, a first-class permanent construction can best be obtained by the "old-rail" method or something similar.



PART OF TRACKAGE FORMERLY CENTERING AT GRAND CENTRAL STATION, NEW YORK CITY.

In planning for a new terminal station adequate to meet all demands of traffic and to accommodate the electric service which was to supplant steam operation on all railroad trains entering or leaving, the engineers were confronted with the problem of depressing and tunneling the great maze of tracks of which a portion only is here shown. This work has been carried out without interfering with street traffic or regular train service.

RAILROAD ENGINEERING.

PART II.

MISCELLANEOUS STRUCTURES.

99. Water Supply. The railroads of the country spend over $6,000,000 per year in supplying water to their locomotives. Part of this expense is due to the fact that a bad quality of water is so injurious to a locomotive boiler (as well as rendering it difficult for the boiler to steam properly) that the added expense of procuring a suitable supply of naturally pure water or of purifying an impure supply is amply justified. A natural water supply is always more or less charged with calcium and magnesium carbonates and sulphates in addition to impurities of almost any nature which come in as the refuse from factories, etc. Some of these impurities are comparatively harmless, especially if the quantity is not large. But the evaporation of the water precipitates the calcium and magnesium, which form deposits on the surface of the boiler.

These deposits are injurious in two ways. In the first place the transfer of heat from the fire to the water is less free and there is thus a waste of energy, and in the next place the metal becomes overheated and perhaps "burned." The safety of the metal of a boiler depends on the free transfer of the intense heat of the fire to the comparatively low heat of the water or steam. The prevention of these deposits may be accomplished in one (or both) of two ways; the frequent cleaning of the boilers through the manholes and handholes provided for the purpose, and by the more or less perfect purification of the water before it enters the boiler.

The location of the water stations must be at such places and intervals as the service demands. There must always be a supply at the extremities of each division and usually at intervals of 15 to 20 miles between. Of course these intervals are varied according to the location of convenient sources of supply. The frequent

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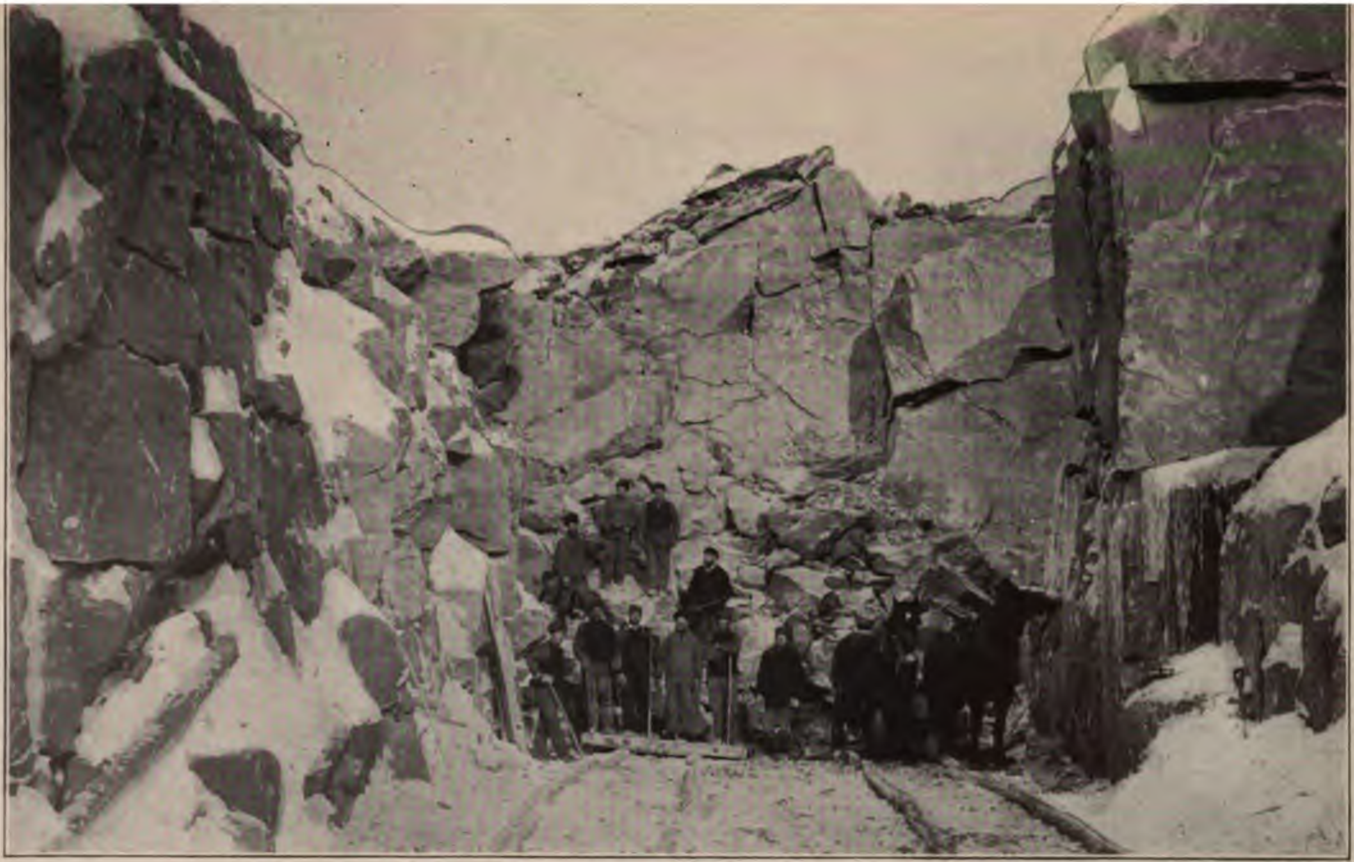
erection of municipal plants for water supply even in small places has led to the utilization of such plants, since a suitable supply for domestic use would be satisfactory for boiler use, and since a reasonable charge to such a large consumer would generally be far less than the cost of maintaining a separate plant. In default of such supplies, a convenient intersecting stream, especially when combined with an existing but perhaps abandoned mill dam which will form a convenient storage reservoir, may be utilized. If the stream passes through a limestone region, the water may become so thoroughly impregnated with calcium compounds that a purifying plant will become a necessity and then there may arise the question of a choice between a conveniently located station with a necessary purifying plant and a less convenient location but a natural supply of purer water.

The chemical purification of water for railroad purposes has become a specialty and must be studied as such. Of course no attempt is made to produce chemically pure water as that would be unnecessarily costly. The reagents chiefly employed are quicklime and sodium carbonate. The lime precipitates the bicarbonate of lime and magnesia in the water. Sodium carbonate gives, by double decomposition in the presence of sulphate of lime, carbonate of lime, which precipitates, and soluble sulphate of soda, which is non-incrustant. The precipitates settle to the bottom of the tank and are drawn off while the purified water is drawn from the upper portion of the tank. Such purification may be accomplished for a few cents per thousand gallons. Still another method of preventing incrustation in the boiler is to introduce directly into the water tank a "non-incrustant" which, as its name implies, will so change the composition of the impurities that they will settle harmlessly and may be readily blown out.

Pumping. Except when water is obtained from a municipal water supply it must be pumped into a tank or reservoir which is usually placed with its bottom 12 to 15 feet above the rails. The pumping may be done with a wind mill, which is very cheap but unreliable, or by an ordinary steam pump operated by a boiler fed with coal, or by a gasoline engine. The last method is becoming very popular, as the pumps require but little attention and the cost of operating them has been found to be as low as one-third or even



A HEAVY ROCK CUTTING

On line of Grand Trunk Pacific Railroad north of Lake Superior.

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one-fourth of the cost of steam pumping. And this is true in spite of the fact that a railroad can usually deliver slack coal or screenings at a pump house alone the line of the road at a cost that may not exceed 30 cents per ton. The cost of pumping to a track tank will usually run at from 2 cents to 6 cents per 1,000 gallons.

Tanks. The construction of the piping from a tank and even of the tanks themselves has become a specialty by manufacturing firms who can make and sell them much cheaper than may be done by any "home-made" method, and, therefore, the details of manufacture need not be here discussed. The tank must be so placed that its nearest face is about 8 feet 6 inches from the track center. When one tank is to serve several tracks or when the supply is taken from a city waterworks, a "standpipe" is necessary. This consists essentially of an upright pipe which stands about 14 feet above the ground where it has a horizontal arm about 7 feet long. This elbow may be turned so that the arm is either parallel or perpendicular to the track. As shown in Fig. 75, the valve mechanism is buried underground and the roof of the pit is protected so that freezing shall be obviated.

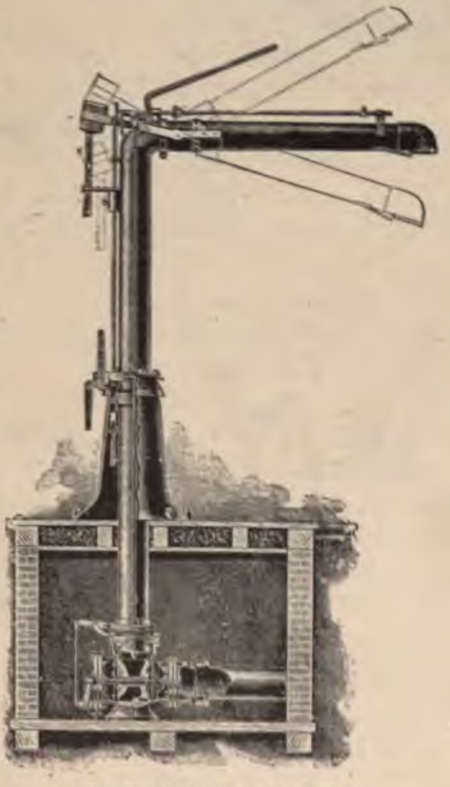


Fig. 75. Automatic Standpipe.

Track Tanks. The demands for high speed require that long runs shall be made without a stop even for water. Very long runs can only be made by taking on water while in motion from a track tank. These have a length of 1,200 to 1,500 feet and must be laid on a stretch of perfectly level track. A large item in the expense of installing such a plant is the cost of the re-grading which is usually necessary to make the track perfectly level. On the ties and midway between the rails is a tank about 19 inches wide, 6 inches deep and as long as desired. This trough will be made of $\frac{3}{16}$-inch steel plate, stiffened and reinforced with angle bars. Such tanks can only be used by engines which are provided with a scoop on the tender which is lowered at the proper time. The high speed causes the water to rush into the scoop with such velocity that it is easily carried to the top of the leader pipe and over into the tender tank. An inclined plane at each end of the trough automatically raises the scoop and when raised it is automatically caught and held so that there is no danger that the scoop shall catch in anything on the track. To prevent the water from freezing in the winter, steam jets should be blown into the water at every 40 to 50 feet of its length. The steam required for this may be many times as great as the steam required for pumping. The cost of such an installation will be upwards of $10,000 and the annual expense about $1,500. Of course these figures will vary with the circumstances.

100. Turntables. The turntable proper is an example in structural engineering which is now almost universally made of structural steel in shops which make such work their specialty. Therefore no discussion will be given of the table. But the table must be supported on a pivot which must have an adequate foundation which must be able to support a load of perhaps 200 tons. The table revolves in a pit which is say 75 feet in diameter and which must have a retaining wall about it. Immediately inside of this wall is a circular track on which rollers on the under side of the turntable may run if the load is eccentric. Since this load on the rail may be large it must have an adequate support.

If the turntable must be located on what is originally sloping ground, the masonry may need to be quite deep and heavy, since the foundation for the pivot should be especially firm. If the subsoil is not self-draining, it should be thoroughly drained by a thorough sub-drainage and the pit should be drained by a pipe leading to a suitable outfall. A turntable is usually located as an adjunct to a roundhouse, but in any case the location should be made so that the switching that must be done before and after

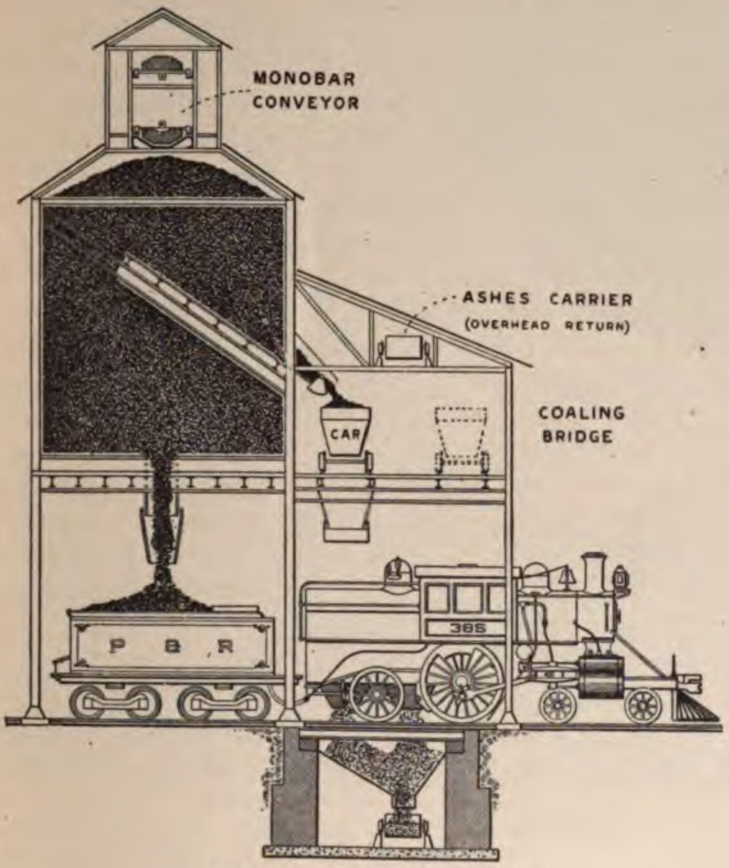


Fig. 76. Coal and Ashes Station.

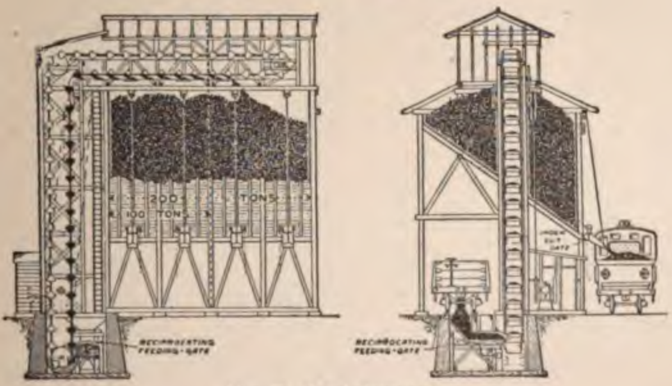


Fig. 77. Coal Conveyor.

using the table shall be made a minimum. The location of the turntable in the yard is an item in the subject of Yards and Terminals.

101. Coaling Stations. The cost of removing ashes from the ashpan of a locomotive to a suitable dumping ground and of supplying the tender with coal may amount to a very considerable item unless special facilities are devised for doing the work cheaply as well as rapidly. Such facilities are especially necessary when the number of locomotives to be taken care of is very great. As shown in Fig. 76, the ashes are discharged directly from the ashpan to a hopper, from which they are discharged into a conveyor which passes over an overhead bridge, to a place where the buckets are discharged directly into a gondola car placed there ready to receive them. By a similar process coal is brought to the plant in gondola cars which dump at the bottom, dumping their load into a conveyor (see Fig. 77), which carries the coal to the top of the coal bin. From the bin it may be discharged directly into the tender or into a conveyor car which will carry it to a locomotive standing on another track.

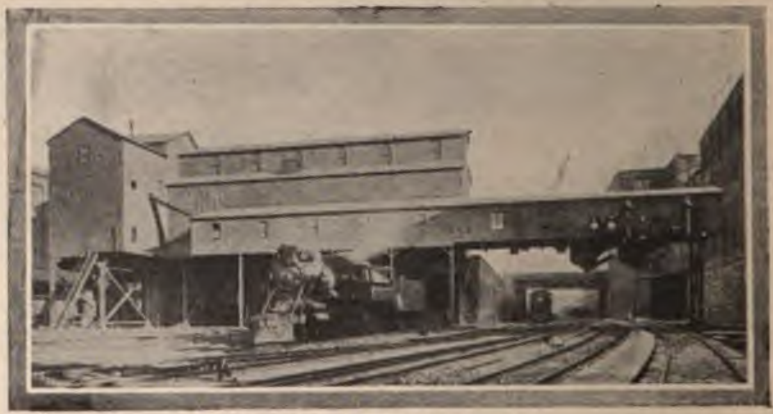


Fig. 78. 1,000-Ton Coaling Station.

Fig. 78 is a general view of the plant shown in detail in Fig. 76. It was erected by the Link-Belt Engineering Co., and while the details are peculiar to that company's work, it may be considered as illustrative of the general method.

102. Engine Houses. On very small roads, where the number of engines to be housed at any one place will never exceed five or six, a rectangular engine house with two or three parallel tracks is the cheapest form of construction. But as the number of engines to be provided for increases, and as space grows more valuable, the "roundhouse" is preferable. Considering the space, tracks and switches required to run a large number of tracks into a rectangular house, the roundhouse will accommodate more engines in proportion to the space required. A turntable is a necessary feature of a roundhouse, but since a turntable would naturally be located at any point on a road where an engine house was required, the cost of the turntable should not be considered as an integral part of the cost of the roundhouse.

Engine houses are used for the minor repairs which continually form a part of the maintenance of any locomotive. Therefore a portion of the tracks should be provided with "pits" or spaces between the rails in which work may be done under the engine. The outer walls are preferably constructed of masonry, although wooden structures are not uncommon on cheaper roads. The roof framing should preferably be of wood, as iron trusses deteriorate very fast under the action of the gases of combustion from the engines. The effect of this is prevented as far as possible by "smoke jacks," which are chimneys suspended from the roof so that they are immediately above the engine stack when each engine is placed where designed. The lower part of this chimney is made adjustable so that it may come down closely over the stack. The smoke jacks are variously made of galvanized iron (very short lived), vitrified pipe (too brittle), cast iron (very heavy), expanded metal and concrete, and even plain wood painted with "fireproof" paint. The floors are best made of brick; cinders are cheap but objectionable, wood is tolerable but lacks durability, concrete is almost an extravagance. Considering that the larger roundhouses may contain locomotives worth several hundred thousand dollars, fire protection is an important feature. One means to this end is the use of rolling steel shutters instead of wooden doors. In Fig. 79 is shown some of the details of what may be considered a typical roundhouse. The figure will illustrate many of the points named above.

103. Cattle Guards. The prevalent opinion that a railroad company is responsible for the death or injury of any cattle which

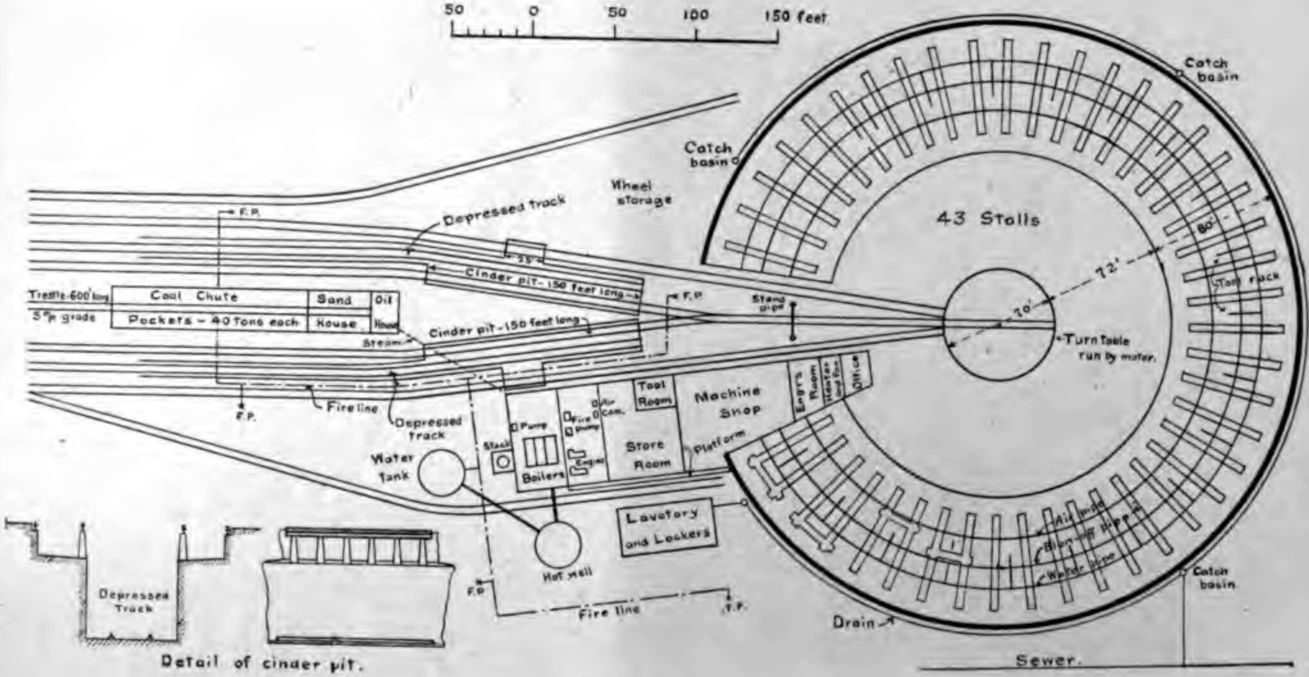


Fig. 79. Details of Typical Roundhouse.

may stray on its right-of-way requires especial precautions that cattle, straying along a highway, shall not turn into the railroad right-of-way. The fundamental idea is a structure which is not an obstruction to trains but over which cattle will not pass. The old way was to use a pit about two feet deep and four feet wide across which the rails were supported on wooden stringers. But this form makes a break in the continuity of the roadbed and is a

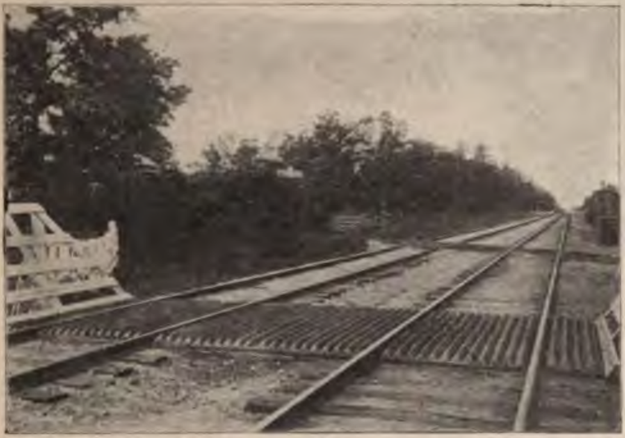


Fig. 80. Climax Cattle Guard.

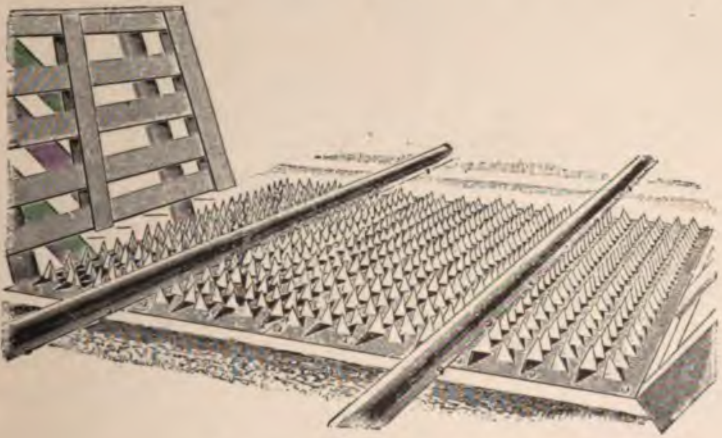


Fig. 81. Sheffield Cattle Guard.

very fruitful source of accidents. This form has, therefore, been definitely abandoned for "surface" cattle guards.

Two forms of these are illustrated in Figs. 80 and 81. The variations in the *surface* adopted are multitudinous. Usually they are made of iron, sometimes of wood and sometimes of some form of tile or cement which is not subject to decay or rust. Any form must have in addition the fences extending from the sides of the right-of-way up to the ends of the ties. These fences will be "headed" by a short guard fence, as shown in the left of each of the figures, which will prevent cattle from stepping over the end of the fence.

TRACK AND TRACK WORK MATERIALS.

104. Ballast. The ideal ballast must transfer the applied load over a large surface; it must hold the ties in place horizontally; it must carry off the rain water and thereby prevent freezing up in winter; it must be such that the ties may be readily adjusted to the true grade line and it must produce an elastic roadbed. The various materials used for ballast fulfill these conditions in variable degrees and at various costs. The most perfect and costly ballast is not necessarily the best for a light traffic road, but on the other hand many light traffic roads are increasing their operating expenses (unconsciously) in a vain attempt to cut them down by using a cheap form of ballast or none at all. The principal kinds used will be stated with a comment on each one.

Mud. This means *no* ballast except the natural soil. Sometimes the natural soil is sandy or gravelly and will make a very good ballast where it occurs, but no matter how good the soil may be in some places, such a quality cannot be depended on to be continuous throughout the line or even approximately so. Considering that a heavy rain will in one day spoil the results of weeks of patient "surfacing" with mud ballast, it is seldom economical to use it if there is a gravel bed or other source of ballast anywhere

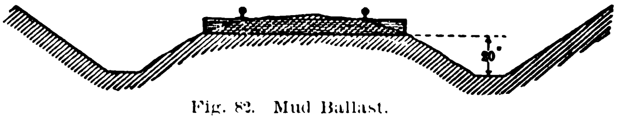


Fig. 82. Mud Ballast.

on the line of the road. If it *must* be used, then the drainage should be exceptionally perfect. The earth should be crowned over the ties in the center and the ditches on each side should be at least 20 inches below the base of the ties. This will facilitate the flow of water to the sides.

Cinders. The advantages are an almost perfect drainage, ease of handling, and cheapness, for, after the road is in operation, their use is but the utilization of a waste product. The chief disadvantage lies in the dust produced as the particles are ground up by use. Incidentally, a light traffic road would require a long time to produce enough ashes to ballast the whole road, which would imply a long period of operation with no ballast at all.

Slag. In certain places such ballast is very cheaply obtained as a waste product, it being given away for the hauling. It is free from dust and the drainage is perfect.

Shells, fine coal, etc. These are only used when their proximity makes them especially cheap. They become dusty in dry weather and correspondingly imperfect in their drainage qualities. They soon become but little better than "mud."

Gravel. A large proportion of the railroad mileage of the country is laid with gravel ballast. This is because gravel beds are so frequently found on the lines of roads, from which the gravel may be dug with a steam shovel, loaded on to cars and hauled to any desired point where it is perhaps unloaded mechanically, the only strictly hand work in the whole operation being the tamping of the ballast in the track. Such methods make the cost per cubic yard very small. The gravel is easily handled and affords almost perfect drainage. If the gravel contains very fine stones or dirt, it should be screened over a half-inch screen to take the fine stuff out.

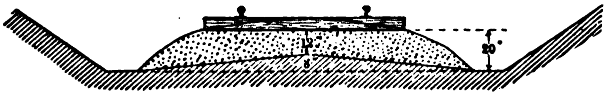


Fig. 83. Gravel Ballast.

Broken Stone. This is the best form of ballast obtainable, and usually the most expensive. Although hand-broken stone is preferable, the cost of machine crushed stone is so much less that it is almost exclusively used. They should be broken so that they will pass through a 1½-inch or 2-inch ring. It is most easily shoveled with forks, and this method has the additional advantage that the finest chips and dirt will be screened out. Such ballast holds the ties more firmly than any other form and hence is almost an essential for roads handling a great and heavy traffic at high speed. For a light traffic road running few trains and these at very moderate speed, the use of rock ballast would be almost a useless luxury unless the broken stone were very cheap and gravel were expensive or unobtainable.

Amount required. Good practice requires a depth of 12 inches of gravel or broken stone under the ties. With 6-inch × 8-inch ties spaced 24 inches between centers, the amount between the ties will be equivalent to an additional depth of about 4 inches.

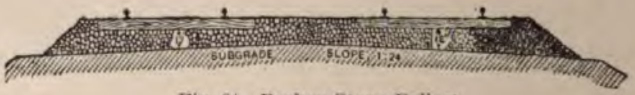


Fig. 84. Broken Stone Ballast.

If the ballast has an average width of 10 feet, say 8 feet at the top and 12 feet at the bottom, then one mile of track will contain 2,607 cubic yards. Broken stone requires a little more than this since there should be a shoulder of ballast on the ends of the ties. (See Fig. 84.)

Method of laying. When ballast is laid during the original construction of the road, the proper method is to haul the most of the ballast with carts or on the contractor's temporary track and spread it evenly to the level of the bottom of the ties. Then the ties and rails can be laid and a construction train can haul whatever ballast is required for surfacing and tamping. When the ties and rails are laid on the bare subsoil and the construction trains with ballast are run over it, the rails are apt to become badly bent and kinked. A compromise between the above methods is to use light construction cars which may run on the standard gauge track without doing the injury that would be caused by standard loaded rolling stock.

Cost. The cost of ballast depends on (*a*) the initial cost as it comes to the road, (*b*) on the distance from the source of supply to the place where used, and (*c*) on the method of handling. A little thought will show the variation in these items for different roads, and therefore any estimates of cost are necessarily approximate. As an *average* figure the cost of broken stone ballast in the track may be computed as $1.25 per cubic yard, and the cost of gravel may be put at 60 cents. The cost of placing and tamping gravel ballast is estimated at 20 to 24 cents, while the similar estimate for cinders is put at only 12 to 15 cents. The cost of loading gravel on cars, using a steam shovel, is estimated at 6 to 10 cents per cubic yard.

105. Ties. The cost of ties to a railroad is too apt to be superficially considered as the mere market price of the ties delivered to the road. The true cost is the cost of the maintenance of suitable ties in the roadbed for an indefinite length of time. The first cost is but one item in the total cost. A cheap tie must be soon renewed. The labor of renewal is a considerable item of cost. The renewal disturbs the roadbed, which requires adjustment to keep it from getting uneven. The unavoidable unevenness of the roadbed has an actual although uncertain effect on operating expenses, increasing the fuel consumption and wear and tear on the rolling stock. It even has some effect on possible or safe speed.

In round numbers, if the cost of buying and placing a good tie is twice that of a cheap tie, and the good tie lasts twice as long as the cheap tie, the economics of the cases are nearly equal. But on the one hand we have the interest on the *extra* cost of the good tie for the lifetime of the cheaper tie and on the other hand we have the additional cost of maintenance of way when using the poorer ties and the indefinite increase of operating expenses due to a poor roadbed. The annual cost of a system of ties should therefore be considered as the sum of (*a*) the interest on the first cost, (*b*) the annual sinking fund that would buy a new tie at the end of its life, and (*c*) the average annual maintenance for the life of the tie, which includes the cost of laying and the considerable amount of subsequent tamping that must be done until the tie is settled in the roadbed, besides the regular track work due to the tie. Such a method of comparison is essential in considering the economics of chemically treated ties and untreated ties.

Wood. A good tie must last as long as possible in the ground, must be hard enough not to be unduly affected by "rail-cutting," must be hard and tough enough to hold the spikes, and finally must be reasonably cheap. Throughout the United States some of the varieties of oak fulfill these conditions (on the whole) better than any other kind. Pine is the second choice, largely determined by its local cheapness. Cedar and chestnut come next, while redwood, cypress, hemlock, tamarack and a few others have a lesser use. Redwood and cypress are as good as any from the standpoint of mere decay, but they are so soft that the rails cut them and spikes have but little holding power. Since spikes must be driven within a very small area on the face of the tie (for the tie must be placed symmetrically under the rails), when a spike is partially pulled up by the rail tending to turn over, the spike must be re-driven very near its former position. On a curve there is a very great force tending to turn the rail over, and when the holding power of the spikes is not very great, they must be frequently re-driven. Forcing them down in the same hole is almost useless. It thus happens that a tie of soft but durable wood will be "spike-killed" long before any decay has set in. Redwood ties have been largely used in the West, and when they are protected by tie plates from rail-cutting, their life in a dry climate is very great, especially on tangents.

Dimensions. Ties for standard gauge roads are 8 feet, 8 feet 6 inches, and occasionally 9 feet in length. They should be 6 inches to 7 inches thick, and if sawed should be 8 inches or 9 inches wide. If they are hewed, they should have a hewed face of about the same amount. Sawed ties are a practical necessity on trestles and bridges, and elsewhere they are preferable. When ties are cut from large timber, as is now frequently the case, sawing is a necessity, but there is a general opinion that hewed "pole" ties are more durable than sawed ties. In any case the bark should be entirely removed before they are laid.

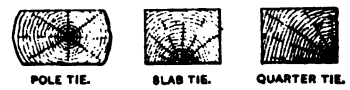


Fig. 85.

Spacing. The most common spacing is 24 inches from center to center, which is the same as 15 per 30-foot rail, which is a common way of stating it. As many as 20 per 30-foot rail are sometimes used if the ties are small, but as this means only 18 inches from center to center, the space left for tamping is small and the support to the rail may be even less than that given by larger ties with wider spacing and more perfect tamping. The spacing should not be exactly even as more support is needed at the joints. Two ties are placed so that the rail joint is evenly supported by them. If the rail joints are "staggered," as is usual, two more joint ties are placed somewhat closer together near the middle of the opposite rail. The remaining ties of the allotment (say 15) per rail will be divided evenly in the remaining spaces.

Rules for cutting. It should be required that hewed ties should have their two faces truly parallel; the trees should be reasonably straight, one rule being that a straight line passing through the center of one end and the center of the middle shall not pass outside of the other end; they must not have severe splits or shakes; they should be cut in winter, or when the sap is down; they should be piled for at least six months before being used. When ties are furnished by farmers along the right of way, it is specified that the ties shall be neatly piled crosswise in piles on ground not lower than the rails, the piles to be at least seven feet from the rails.

Rules for laying and renewing. The largest and best ties should be reserved for joint ties. Whenever spikes are drawn out, the hole should be plugged with a wooden plug which will prevent water from settling in the hole and thus causing rapid decay. Ties should always be laid at right angles to the rail and never obliquely. When renewals are to be made, the requisitions are to be based on an actual count of ties to be renewed and not as the result of any wholesale estimate. It is unwise to use a mixed variety of ties in the track so that their size, elasticity and durability are very different. This will, by the variation in elasticity, cause rough riding.

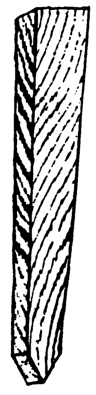


Fig. 86. Wooden Tie Plug.

Cost. Local circumstances very greatly affect the cost, even for the same class of ties. Railroads sometimes succeed in monopolizing the tie production in the territory through which they run by refusing to haul ties for any other customer or railroad, except at prohibitory rates, and control the price somewhat by refusing to pay more than the lowest limit at which the local people will supply the ties. The best ties procurable in a section can thus be procured for 45 to 50 cents per tie, and where common labor is very cheap this price is cut even to 25 cents. On the other hand, the very best of large oak ties will often cost 75 to 80 cents. In view of the above variation in price, any estimates must depend on local conditions.

106. Rails. The form of rail section popularly known as the A.S.C.E. section, was adopted by a committee of the American Society of Civil Engineers in 1893, after a great deal of discussion and study. That form is now used by the most of the railroads of the country. The essential features of the design are that the head shall have 42 per cent of the area of the section, the web 21 per cent and the flange 37 per cent. The top of the head must have a radius of 12 inches; the top corner radius of the head must be $\frac{5}{16}$-inch; the lower corner radius of the head must be $\frac{1}{16}$-inch; the radius of the corners of the flanges must be $\frac{1}{16}$-inch; the radius of the side of the web must be 12 inches, and the radii of the web corners must be $\frac{1}{4}$-inch; the angles with the horizontal of the under side of the head and the top of the flanges are both 13°; the sides of the head are vertical. All of the above dimensions are constant for all sections regardless of the weight of the rail. Fig. 87 shows all of these constant dimensions and indicates by letters the variable dimensions which are given in the following tabular form:

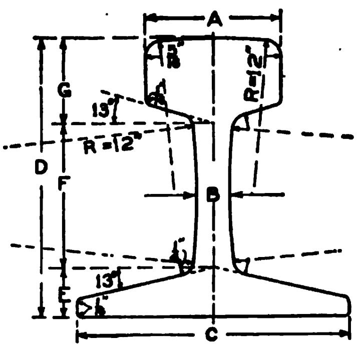


Fig. 87. Am. Soc. C. E. Standard Rail Section.

Dimension, in Inches.	Weight per yard in pounds.												
	40	45	50	55	60	65	70	75	80	85	90	95	100
A	$1\frac{7}{8}$	2	$2\frac{1}{8}$	$2\frac{1}{4}$	$2\frac{3}{8}$	$2\frac{13}{32}$	$2\frac{7}{16}$	$2\frac{15}{32}$	$2\frac{1}{2}$	$2\frac{9}{16}$	$2\frac{5}{8}$	$2\frac{11}{16}$	$2\frac{3}{4}$
B	$\frac{25}{64}$	$\frac{27}{64}$	$\frac{7}{16}$	$\frac{15}{32}$	$\frac{31}{64}$	$\frac{1}{2}$	$\frac{33}{64}$	$\frac{17}{32}$	$\frac{35}{64}$	$\frac{9}{16}$	$\frac{9}{16}$	$\frac{9}{16}$	$\frac{9}{16}$
C & D	$3\frac{1}{2}$	$3\frac{11}{16}$	$3\frac{7}{8}$	$4\frac{1}{16}$	$4\frac{1}{4}$	$4\frac{7}{16}$	$4\frac{5}{8}$	$4\frac{13}{16}$	5	$5\frac{3}{16}$	$5\frac{3}{8}$	$5\frac{9}{16}$	$5\frac{3}{4}$
E	$\frac{5}{8}$	$\frac{21}{32}$	$\frac{11}{16}$	$\frac{23}{32}$	$\frac{49}{64}$	$\frac{25}{32}$	$\frac{13}{16}$	$\frac{27}{32}$	$\frac{7}{8}$	$\frac{57}{64}$	$\frac{59}{64}$	$\frac{15}{16}$	$\frac{31}{32}$
F	$1\frac{55}{64}$	$1\frac{31}{32}$	$2\frac{1}{16}$	$2\frac{11}{64}$	$2\frac{17}{64}$	$2\frac{3}{8}$	$2\frac{15}{32}$	$2\frac{35}{64}$	$2\frac{5}{8}$	$2\frac{3}{4}$	$2\frac{55}{64}$	$2\frac{63}{64}$	$3\frac{5}{64}$
G	$1\frac{1}{64}$	$1\frac{1}{16}$	$1\frac{1}{8}$	$1\frac{11}{64}$	$1\frac{7}{32}$	$1\frac{9}{32}$	$1\frac{11}{32}$	$1\frac{27}{64}$	$1\frac{1}{2}$	$1\frac{35}{64}$	$1\frac{19}{32}$	$1\frac{41}{64}$	$1\frac{45}{64}$

The most important feature in the above design, or the feature which has excited the most discussion, is the radius of the upper corners of the head. Rail wear begins there and rails with sharp corners will wear longer than those with larger radii. The rapidity of the rate of rail wear after the corner has worn off is one proof of this, and so from the maintenance of way standpoint sharp rail corners are desirable. But excessively sharp rail corners produce excessive wear on the flanges of the wheels, not only wearing out the wheels quickly but even rendering them dangerous and liable to cause a derailment. And therefore a compromise between the extremes of sharp rail corners and those of large radii was made by the adoption of the radius $\frac{5}{16}$-inch.

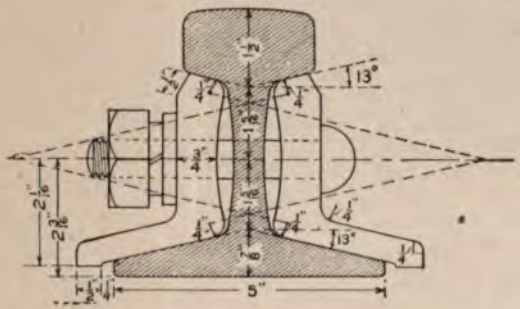


Fig. 88. Rail Joint.

Weight. The weight of rail that should be used on any road is an exceedingly important financial and technical question. It is the largest single item of expenditure in the construction of a road, and the temptation to cut down the item by 5 per cent or 10 per cent is very great. For all ordinary sizes the price per ton is uniform, and therefore a reduction in weight per yard means a corresponding reduction in the cost. But it should be considered that what is desired is a rail that has *stiffness* and *strength*, no matter how much it weighs.

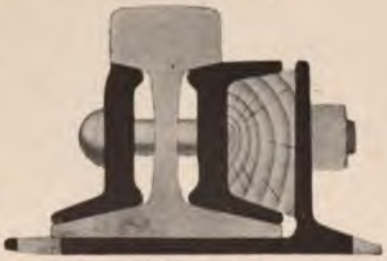


Fig. 89. Weber Rail Joint.

It can readily be proved that if all sizes of rails had exactly similar cross-sections (which is nearly true) then the stiffness of a rail varies as the *square* of the weight and the strength varies as the $\frac{3}{2}$ power. This means that if we add 10 per cent to the weight (and therefore to the cost) of the rail we are adding 21 per cent to the stiffness, and over 15 per cent to the strength. As a more concrete example, suppose that some desire to make the weight of the rail for a road 60 lb. per yard, and others wish to use a 70-lb. rail. At $30 per ton (of 2,240 pounds) the *difference* of cost will be $471.42 per mile of single track. But on the other hand, although the cost is increased by $16\frac{2}{3}$ per cent, the strength is increased 26 per cent, and the stiffness is increased 36 per cent. The increase in stiffness is more than double the increase in cost. Unfortunately there is no absolute criterion as to the amount of stiffness or strength required since it depends largely on the unknown, uncertain and variable tamping of the ties and the support which the ties receive from the ballast. But the above *relative* figures hold good, and considering that a stiff track means decreased rolling resistance, higher speed and greater safety, a considerable increase in weight over that minimum on which it would be possible to run trains is not only justifiable but is a measure of true economy. As a general statement, it may be said that 60 lb. per yard is the lightest

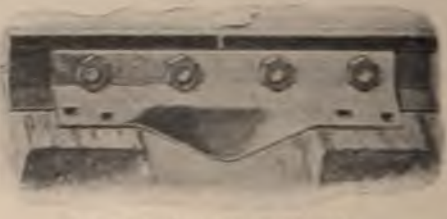


Fig. 90. Bonzano Rail Joint.

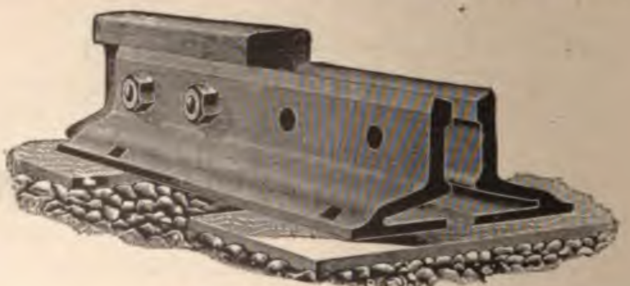
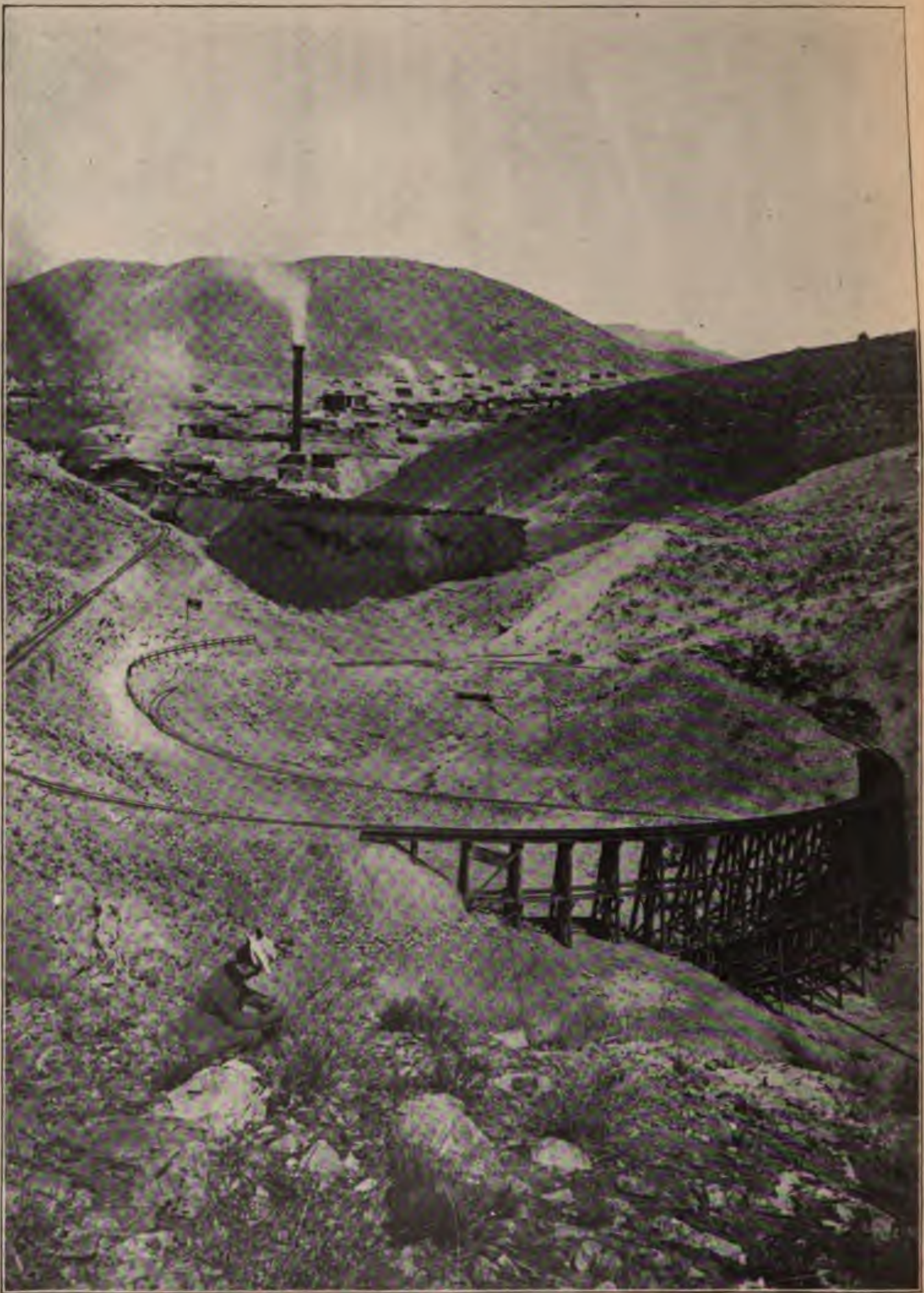


Fig. 91. Continuous Rail Joint.



ODD CURVE ON THE OROYA LINE, PERU

Copyright, 1907, by Underwood & Underwood, New York weight which should be used on a standard gauge road running ordinary rolling stock, no matter how light the traffic. Roads with a fair business should have 70-lb. rails. The great trunk lines are relaying with 100-lb. rails on the heavy traffic divisions, and usually have as heavy as 85-lb. rails on all but the small branches.

Length. The standard specifications proposed by a committee of the American Railway Engineering and Maintenance of Way Association in 1902 contained this clause: "The standard length of rails shall be 33 feet. Ten per cent of the entire order will be accepted in shorter lengths, varying by even feet down to 27 feet. A variation of $\frac{1}{4}$-inch in length from that specified will be allowed."

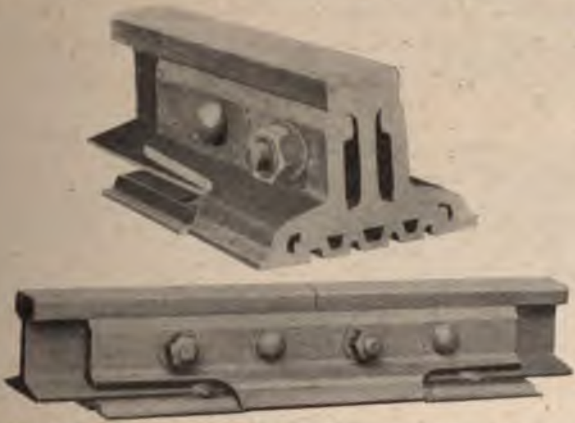


Fig. 92. Wolhaupter Rail Joint and Section Through Center.

During late years much experimenting has been done with the idea of increasing the length of rail, and a considerable amount of rails of 45 and even 60 feet has been laid. These have the undoubted advantage of saving a proportionate number of rail joints, which are always a source of trouble, but at the same time the allowance for expansion which must be made at every joint must be proportionately increased. The above recent standard specification apparently indicates that the increase in length has not proven desirable.

107. Rail Joints. The action of a heavy wheel rolling on an elastic rail is to cause a wave of elasticity to run in front of the point of contact. A perfect track is one that will keep that wave of elasticity perfectly uniform, which requires that the rail joint should have the same strength and stiffness as the rail. Only a welding of the rails, making them continuous, would accomplish this. Any rail joint which is as strong as the rail is necessarily much heavier and stiffer. Passing by the older forms which have now become obsolete, we have in Figs. 88 to 93 the forms which are now competing for adoption. The "angle bar" is still used more than any other kind, but many of the other forms have demonstrated their reliability and fulfilment of the requirements as nearly as may be hoped for. Nearly all of these designs are used exclusively as "suspended" joints rather than as "supported" joints, the difference being, as the name implies, that a suspended joint is placed between two ties so that each end of the joint has an equal bearing on the ties; a supported joint is set directly over a tie and hence must get practically its whole support from that one tie, unless the joint is so long that it rests on the adjacent ties, thus making it a "three-tie" joint.

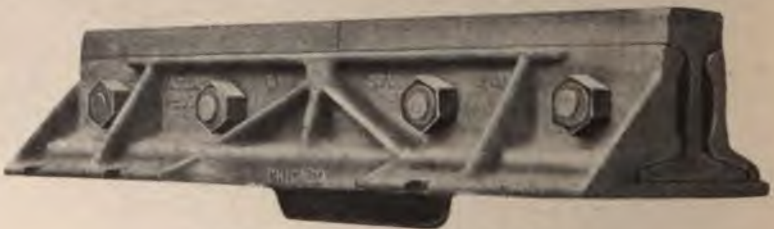


Fig. 93. Atlas Suspended Rail Joint.

Angle bars are usually about 26 inches long. Of course, the bars, of whatever kind, should be so made that they will fit closely under the head of the rail and also have a close fit on the top of the flange. This means that every rail joint must be made with special reference to the particular design of rail with which it is to be used and that it will fit no other design. For the smaller sizes of rails and on light traffic roads, four-bolt angle bars are used, but the longer and heavier bars are usually made with six holes. The holes are made in a somewhat elliptical form and the track bolt has a corresponding form immediately under the head. The bolt is thus prevented from turning when the nut is screwed on or off. The holes in the rail are made about $\frac{1}{4}$ inch larger in diameter than the bolt. This is to allow room for expansion of the rail due to temperature.

Insulated Joints. Rails are very frequently used to form an electric circuit as part of the system of signaling. As an item in the system it is required that certain joints shall be so made that no current shall pass between adjacent rails. This requires the use of insulated joints. A plate of some insulating material is placed between the ends of the rails and even the joint bars, of whatever kind, must be made of wood, or if of metal must have the metal insulated from the rails. One form of such a joint is illustrated in Fig. 94.

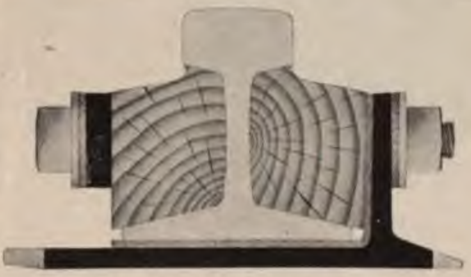


Fig. 94. Insulated Joint for Track Circuit.

108. Tie Plates. Many of the soft-wood ties are very durable as regards decay, but are "cut" by the rail very badly. This is not due to mere static pressure but to the working of the rail on the tie during expansion, and to impact when the rail has become loosened somewhat from the tie and a wheel load suddenly forces it down with a hammer blow. The cutting on curves is also due to the excessive pressure produced by the edges of the flanges which is developed by the centrifugal action of the rolling stock. Another advantage in the use of tie plates lies in the fact that the spikes are mutually supported; a spike cannot be forced laterally in the tie without drawing the tie plate with it and this is resisted by all the spikes passing through the tie plate. The cost is insignificant compared with the added life of the tie, especially if it is a soft wood tie. The advantage with an oak tie is not so great proportionally.

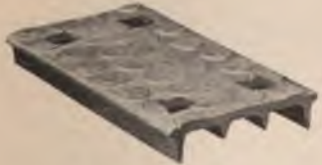


Fig. 95. Tie Plate.

It is very important that the spikes should fit the spike holes with but very little play, otherwise one of the primary objects of the plate will be defeated, and the rail will not be secure against lateral motion. Note that the flanges on the lower side of the plate not only stiffen it and make it much stronger structurally but they also secure the plate to the tie and prevent an objectionable rattling. The very presence of these flanges, however, requires that the plates shall be driven on to the tie.

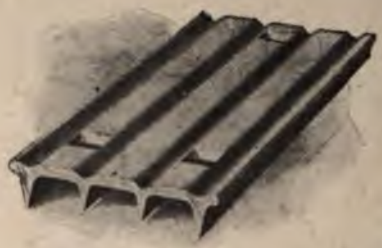


Fig. 96. Wolhaupter Tie Plate.

A common but objectionable method of doing this is to put the plates in place under the rails, drive the spikes as far as they will go and then depend on the passage of the first trains to force the plates into place. If the spikes are immediately driven home as soon as the plates are down, the method is not so objectionable, but even a temporary delay will result in loose spikes and rails. A very heavy wooden maul is the best ordinary tool. The very best results are obtained by utilizing a pile driver, setting the plates accurately to gauge and then driving them home by a single blow from the pile hammer, allowing it to fall two or three feet.

109. Rail Braces. The pressure against the outer rail on a curve, and also the pressure against the inner rail when a train stops on a curve which has a considerable super-elevation, is frequently provided for by "rail braces" such as are illustrated in Figs. 99 and 100. Sometimes these are made of cast iron, but these are brittle and are apt to be broken by a blow from a spike maul when the spikes are driven. The preferable form, although it is more expensive, is to forge them or "press" them from wrought iron or steel. In Fig. 100 is shown a form which has a plate which runs under the rail which thus makes it a combined rail brace and tie plate.

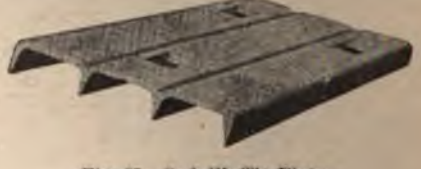
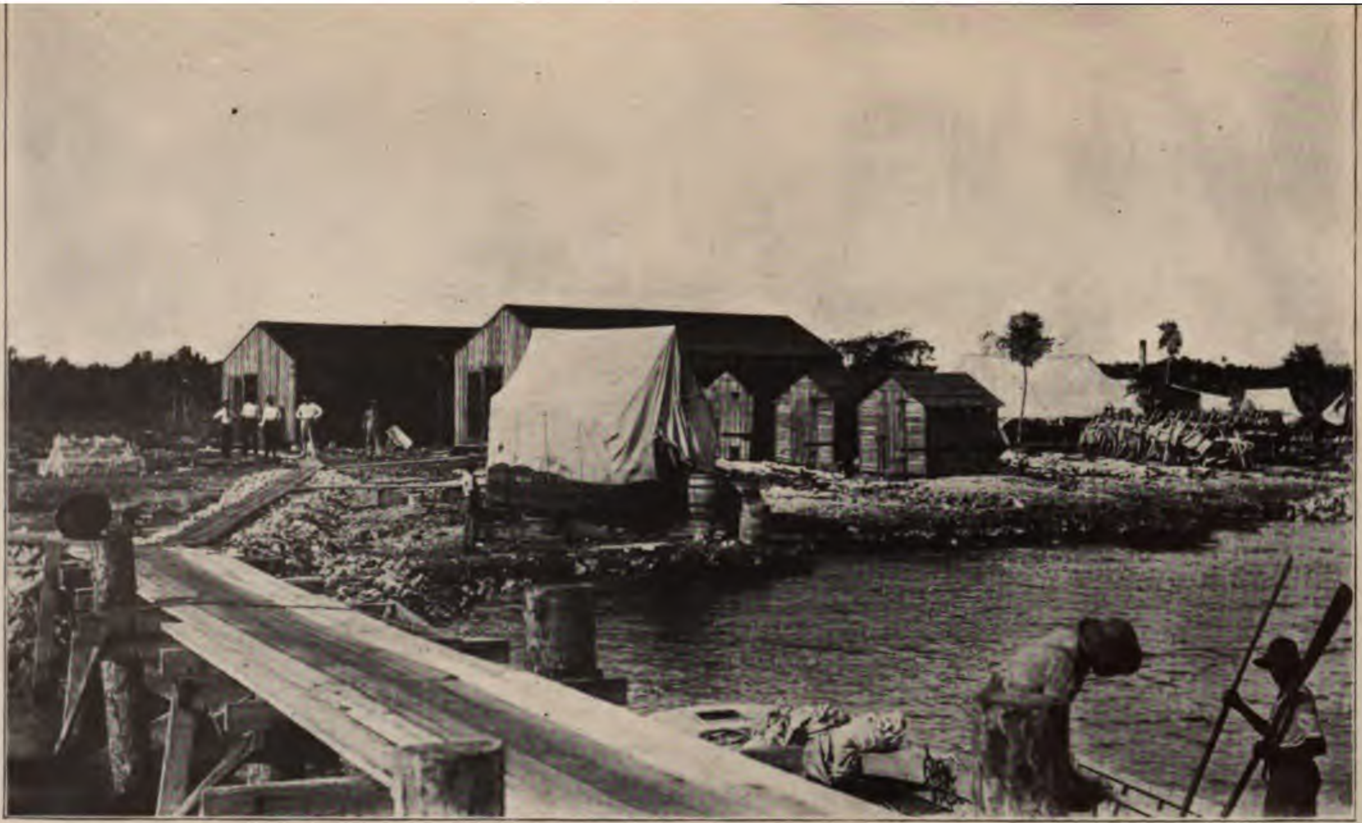


Fig. 97. Q. & W. Tie Plate.

110. Spikes. The fundamental requirement of a spike is holding power, but it must also be cheap, easily applied and easily removed when necessary. Many devices, such as making the surface rough and even jagged, have been tried to increase the hold-



RAILROAD CONSTRUCTION CAMP IN FLORIDA
Key West extension of Florida East Coast Railway.

ing power. But it has been found that the destruction of the fibre of the wood caused by these devices has actually reduced the holding power and that the best form is a spike with plane and smooth faces. The point should be made so as to *cut* the fibres of the wood instead of crushing them. By this means the fibres are pressed outward and downward, and thus any upward pull only tends to draw the fibres back to their original place and so increase the pressure against the spike and thus increase the friction and the holding power. The standard spike for rails weighing more than 56 pounds per yard is 5½ inches long and $\frac{9}{16}$-inch square. There will be about 375 in a keg of 200 pounds. On this basis, if the ties are 24 inches apart and four are used per tie, there will be required 5,632 spikes per mile or 28.16 kegs. Of course a consider-

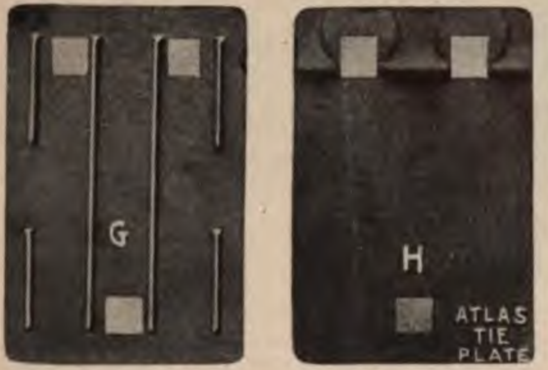


Fig. 98. Atlas Tie Plate.

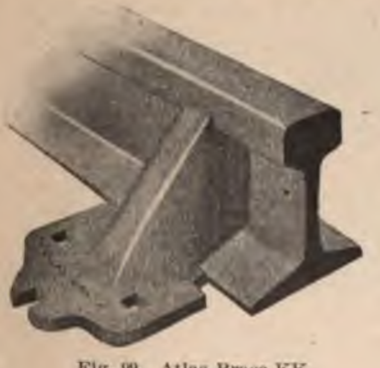


Fig. 99. Atlas Brace KK.

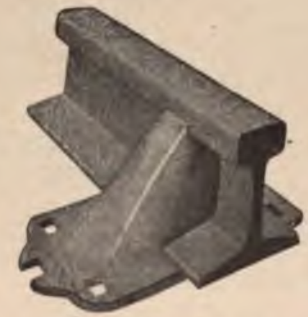


Fig. 100. Atlas Brace K.

able allowance must be made for loss and waste of various kinds.

111. Track Bolts. The track bolt must have sufficient strength to hold the angle plates together with such force as will develop the full strength of the angle plates. And yet this must be accomplished so that the friction developed will not be so great that the rails may not slide in the joints during temperature changes. On a straight track the contractive pull due to a fall of temperature is so great that no possible gripping of the bolts could prevent slipping, but it is quite possible that when rails expand, and especially when on a curve, the resistance to slipping might be so great that the track would bulge out of alignment instead of slipping at the joints. Such an effect does actually take place when the allowance fcr expansion is insufficient and the rails continue to expand after they have butted end to end.

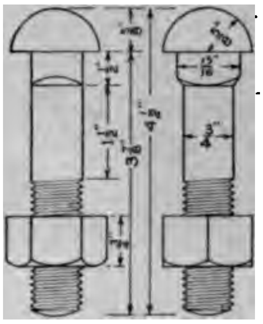


Fig. 101. Track Bolts.

Another requirement is that the bolts shall not turn while the nut is being turned. This is accomplished by an enlargement of the bolt just under the head, as shown in Fig. 101. This fits fairly closely in a corresponding oval-shaped hole in the angle plate. The sizes shown in the figure are about what should be used with a 70 or 80-pound rail. Heavier rails require a longer bolt and one that is proportionately heavier. The type of rail joint used, and also the type of nut lock if any, will determine the required length of bolt, while the weight of rail should determine the diameter. The diameters vary from $\frac{3}{4}$ inch to 1 inch, and the lengths from 3 inches to 5 inches.

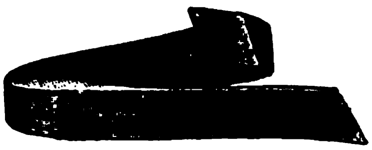


Fig. 102. Ajax Tail Washer.

112. Nut Locks. There are three types of nut lock (*a*) those which have an elastic cushion under the nut which absorbs the vibrations that would otherwise loosen the nut, (*b*) those by which the nut is made to grip the bolt (by some unusual device) so that vibration will be insufficient to loosen it, and (*c*) the "positive" type, in which the locks are prevented from turning by some definite and positive mechanical check.

The "Ajax Tail Washer," shown in Fig. 102, is a sample of the first class, although it also has some of the elements of the third class, since the sharp steel points will tend to bite into both the under side of the nut and the side of the angle plate where it rests whenever there is a tendency for the nut to turn backward. These points merely drag and slip when the nut is being tightened.

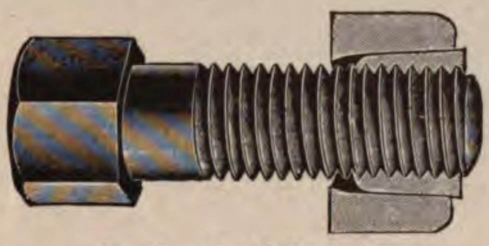


Fig. 103. Columbia Nut Lock.

The Columbia nut lock, shown in Fig. 103, is a sample of the second class. The nut is compound, the inner piece being a four-sided frustum of a pyramid, the edges being rounded. This fits into a corresponding recess in the outer piece. The inner piece is also cut through so that it may be slightly squeezed together. The pyramidal form requires both pieces to turn together. When the outer piece comes in contact with the angle plate it is forced back (relatively to the inner piece) which squeezes the inner piece together and causes it to grip the bolt. The more the nut is turned, the tighter the grip.

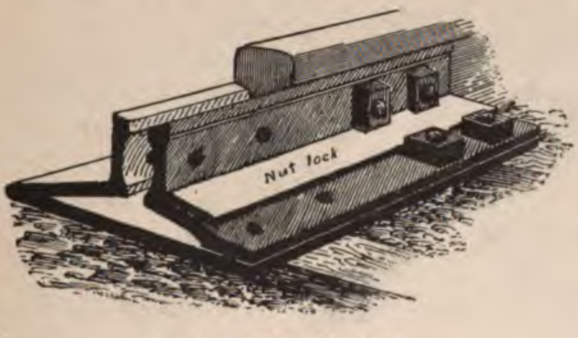


Fig. 104. Gordon Nut Lock.

The Gordon nut lock, shown in Fig. 104, is a sample of the third class, although it is designed to be used only with the form of angle plate which is shown. In the form shown the square nuts must be turned until one edge is exactly on line. A one-eighth turn forward or back will always accomplish this. Thus when the bar is slipped in all nuts are absolutely prevented from turning. The above designs have been selected as mere samples of each class from a great multitude of designs of greater or less merit which are on the market.

LAYING TRACK.

113. Surveying. After the earthwork is completed and the culverts and bridges are built, the center line of the track must be re-located on the roadbed surface of the fills and cuts. Reference points should have been established during the original survey so that by the intersection of two radii swung from permanently established points the beginnings and endings of all curves may be re-located. Then all intermediate stations should be filled in. A line of levels should then be run and the agreement of these levels with the designed grade should be determined. If the levels of the cuts and fills has been followed with sufficient closeness during construction, there should be no discrepancy except that the levels of fills should be somewhat higher than that called for so as to allow for subsequent settlement.

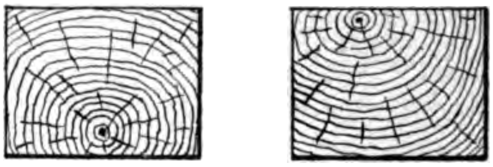

RIGHT. WRONG.

Fig. 105. Right and Wrong Method of Laying Ties.

114. Laying Ballast. This has already been discussed in §104, as has also the policy of laying the ties and rails first and then drawing the ballast in a construction train on the poorly supported track.

115. Laying Ties. If the ties have been sawed to an exact length, the alignment of one end will of course line up the other but when ties have been hewed and chopped off and sometimes even when they have been sawed, there is a range of several inches in their length and then it is required that they shall be aligned at one end or the other. A little stick may be furnished the trackmen as a spacer, but with a little experience they will space the ties as closely to the required spacing as is necessary. The ties should always be laid with rings convex upward rather than concave. Of course a pole tie, when it is perfectly symmetrical, will be the same either way, but there is usually a choice, as is shown by the figure. When the rings are concave upward there is a greater chance for water to soak in and cause decay. Turning the tie the other way, the water will shed off more freely.

116. Laying Rails. Rails should be laid so that the joints are staggered as nearly as possible. This requires a half-rail length at the start. But the difference of length of the outer and inner rails of a curve will disturb the arrangement of the joints, no matter how perfectly it may start. These differences may be neutralized by selecting rails which are a foot or two shorter than the usual length. But the occurrence of a switch will require a readjustment of the joints, and may require a rail cutting so as to bring a joint where desired. Very short lengths of rail should be avoided. If a full length rail comes a few feet short of a point where a joint *must* be made, it should be cut so that both pieces shall have a fair length. The rails are first laid approximately in position and end to end.

When placing the joints on the rails, allowance must be made for rail expansion due to temperature. The theoretical amount to be allowed is .0000065 of the length for each degree Fahrenheit. If it could be readily determined just what is the temperature of the rail (which is possibly much higher than that of the air) at the time the rail is laid and also the highest and lowest temperature that it will ever attain, the problem would be comparatively simple, but the fact that these quantities are so uncertain seem to render useless any attempt at an exact calculation and to justify the rough and ready rule of "allowing $\frac{5}{16}$ inch for coldest weather, $\frac{1}{8}$-inch during the spring and fall, and $\frac{1}{16}$-inch during the very hottest weather." The allowance of $\frac{1}{16}$-inch during the very hottest weather is apparently based on the idea that the rails should never be allowed to butt up against each other, for then any additional expansion will cause the rails to buckle. If a rail was laid when its actual temperature was 60 F., its length of 33 feet would be increased by about $\frac{1}{8}$ inch if its temperature were raised to

120°, as might readily happen under a burning summer sun when the temperature of the air in the shade was perhaps 100°. A practical method of making an allowance which would be sufficiently accurate would be as follows: Place a bulb thermometer (one without a metal frame) so that the bulb lies against the rail and then cover it up so as to protect it from the air and so that it will assume the temperature of the rail as closely as possible. The expansion of a 33-foot rail for each degree is

$$.0000065 \times 33 \times 12 = .002574 \text{ inch.}$$

If we allow 120° (some allow 150°) as the maximum beyond which it is assumed that the temperature will never rise, then the difference between this maximum and the ascertained temperature of the rail, when multiplied by the above allowance per degree, equals the gap to be allowed at each joint. Strips of sheet metal of the required thickness should be furnished to the trackmen. These strips are placed temporarily between the rail ends which obviates any necessity for measuring on their part. When the joints have been bolted up, one line of rails is spiked so that they are at the proper distance from the ends of the ties. Then by using a "track gauge" at every other tie the other line of rails may be spiked down. The intermediate ties are then spiked. "Standard" gauge, which is in almost universal use in this country, is 4 feet 8½ inches = 4.708 feet. Although the gauging should be all right for these other ties, the gauge should be at hand to check the previous work, especially if it is on a sharp curve. Track instructions frequently specify that rails should be previously bent before laying around curves, or in other words, that the rails should have the proper curve when lying freely on the ties. Of course the necessity for this increases with the degree of curvature, it being unnecessary for very easy curves.

The practical trouble comes at the joints; the rails instead of having a common tangent will intersect at an angle which is destructive both to the track and the rolling stock when trains are run at high speed. The ideal method is to have the rail bending done by rollers in a rolling mill and this method is almost a necessity for the very sharp curvature employed on some electric roads. The field method is to use a "rail bender" which bends the rail in lengths of about two feet and which must be operated very carefully and skilfully to avoid ruining the rail. A rail is bent until, when a string is stretched from the inside of the head at one end to the inside of the head at the other end, the distance from the middle point of the string to the inside of the head at the middle of the rail is as computed below:

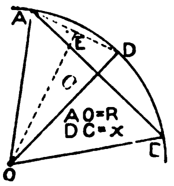


Fig. 106.

In Fig. 106, since the triangles AOE and ADC are similar, AO : AE :: AD : DC, or $R = \frac{1}{2} AD^2 \div x$. When as is usual, the arc is very short compared with the radius, $AD = \frac{1}{2} AB$ very nearly. Making this substitution, we have

$$R = \frac{\text{chord}^2}{8x} \text{ (very nearly)} \qquad (54)$$

Inverting the formula we have the formula required for present use:

$$x = \frac{\text{chord}^2}{8R} \text{ (very nearly)} \qquad (55)$$

Although not mathematically accurate, the maximum error in any practical case is far within the attainable accuracy using a string.

Example. What should be the middle ordinate for the outer rail (33 feet long) for a 6 degree curve? We will call the chord 33 feet since the slight inaccuracy involved only tends to neutralize the inaccuracy of the formula. $R = 955.37 + 2.35 = 957.72$. Then 33^2 (which equals 1089) divided by $(8 \times 957.72) = .142$ foot or 1.70 inches. If a similar calculation is made for the inside rail the difference in the ordinate is less than .01 inch, which shows that unless the curvature is excessively sharp there is no need to make the allowance for half-gauge (2.35, as is done above) nor even to use great accuracy in the decimals. A table giving the middle ordinates for 33-foot rails for different degrees of curvature is a desirable part of the equipment of each track foreman.

The spikes on the opposite sides of a rail should be driven "staggering," so that there will be less tendency to split the tie. The direction of the staggering should be reversed at the two ends of the tie, so as to prevent a loosening of the hold of the spikes, such as would occur if the reverse method were used and the tie were to become fastened in the ballast. Such an item of construction, while very simple, is of vital importance.

117. Surfacing. Track centers (stakes) having been placed in line, the alignment of the track is made perfect. The rail laying should have been done with the rails a few inches below their proper grade. Then jacks are placed under the ties (or rails, as most convenient) and the track is raised to grade, as given by grade stakes which should have been previously set. Using tamping picks or shovels, the ballast is jammed under the ties until they are solid at the desired grade. Picks or tamping bars are best for tamping broken stone ballast, but gravel can be most easily tamped with shovels.

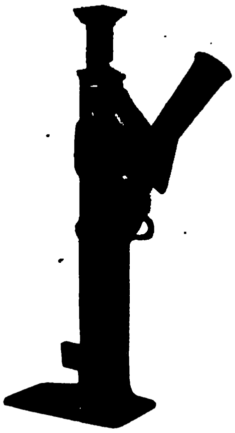


Fig. 107. Trip Ballast Gang Jack

118. Super-elevation of the Outer Rail on Curves. It is one of the demonstrations of physics that the force required to make a mass move in a circular path equals $Gv^2 \div gR$, in which G is the weight, v the velocity in feet per second, g the acceleration of the force of gravity in feet per second in a second, and R the radius of curvature. If the rails on a curve were level transversely, such a force could only be furnished by the pressure of the wheel flanges against the rail. To avoid this objectionable pressure, the outer rail is elevated until the inward component of the inclined wheel pressure equals the computed centripetal force required.

In Fig. 108, ob may represent the resultant pressure on the rails at the same scale at which oc represents the weight G. Then ao is the required centripetal force. From similar triangles, we may write $sn : sm :: ao : oc$. Call $g = 32.17$. Call $R = 5730 \div D$, which is sufficiently accurate for the purpose. Call $v = 5280V \div 3600$, in which V is the velocity in miles per hour. mn is the distance between rail centers, which for an 80-lb. rail and standard gauge is 4.916 feet; sm is slightly less than this. As an average value, call it 4.900, which is its exact value when the super-elevation is $4\frac{3}{4}$ inches. Calling $sn = e$, we have

PART OF THE TRACKAGE TO BE ELEVATED IN THE CITY OF CHICAGO

Yard of the Chicago & Western Indiana Railroad between 51st and 47th streets. The city contains within its limits (1908) over 1,600 miles of steam road tracks. All street crossings at grade are being abolished by the work of track elevation carried out by the railroads themselves, without any expense to the city.

$$e = sm\,\frac{ao}{oc} = 4.9\,\frac{Ge^2}{gR}\,\frac{1}{G} = \frac{4.9 \times 5280^2V^2D}{32.17 \times 3600^2 \times 5730}$$

$$e = .0000572V^2D \qquad \textbf{(56)}$$

Studying the above formula, it will first be noticed that the required super-elevation varies as the *square* of the velocity, which means that a change of velocity of only 10 per cent would require a change of super-elevation of 21 per cent. Since train velocities over any road are so very variable, it shows that it is impossible to make any super-elevation fit all trains even approximately. There are several approximations in the above formula, but none of them will affect the result as much as a change of less than one per cent in the velocity.

Practical Rules. A very simple and commonly used rule is to elevate one inch for each degree of curvature. This rule agrees with the above formula when the velocity is about 38 miles per hour. If a train is running slower than the speed for which the super-elevation was designed, the practical effect is to relieve the pressure against the outer rail which still exists in spite of super-elevation on account of the necessity of turning the groups of four or six wheels under a truck or engine. Therefore the better plan is to elevate for the fastest trains. Thirty-eight miles an hour is so near the maximum for a light traffic branch line, that the above rule is very fair, although, of course, not so good as a more accurate one.

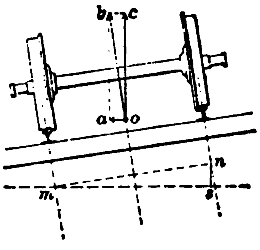


Fig. 108.

Another rule, which is especially good for track maintenance when the track foreman may not even know the degree of curve, is developed as follows: Assume that x in equation 55 is equal to e in equation 56, and we have

$$\frac{chord^2}{8R} = .0000572\ V^2D$$

but since $D = 5730 \div R$, we have

$$chord^2 = 2.621\ V^2 \text{ and}$$

$$chord = 1.62\ V \qquad \textbf{(57)}$$

Assume that the limit of 50 miles per hour is set as the speed of the fastest trains, then *chord* = 1.62 × 50 = 81 feet. This means that if a string or tape, having a length of 81 feet, is stretched between two points at that distance apart on the inner head of the outer rail, the length of the ordinate at the middle of the string equals the required super-elevation for 50 miles per hour. Similar computations can be made and tabulated for all other desired speeds. On double track, since the speed on an ascending grade will almost certainly be less than the speed of trains coming down that grade, there should theoretically be a difference in the super-[illegible] allow for this difference of speed. On some roads [illegible] instructions contain specific instructions to allow for this.

SWITCHES A[illegible]TURNOUTS.

119. Switch Constr[illegible]**n.**—[illegible]he universal method of keeping the wheels of railroad [illegible] stock on the rails is to put flanges on the inner edges of the wheels. When the wheels are to be led away from the main track, it must be done by creating a new pathway for these flanges. This is done by leading the wheel flanges *through* the rails or by raising the wheels sufficiently so

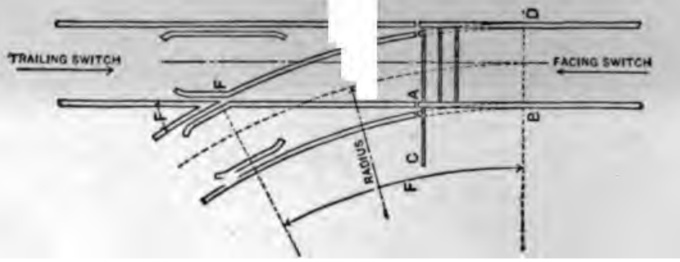


Fig. 109. Stub Switch.

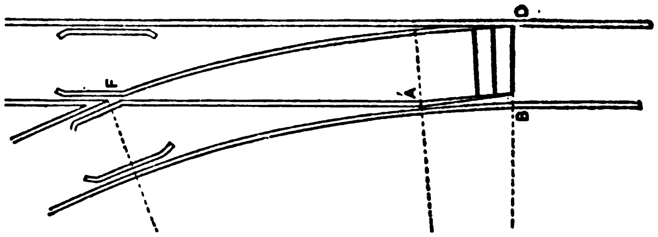


Fig. 110. Point Switch.

that they may pass *over* the rails. Both methods will be described. The method of leading the flanges through the rails is most commonly used since it does not require *raising* the rolling stock over the rail.

When the rails are first led out from the main track, it must be done by one of two general methods, the stub-switch method, illustrated in Fig. 109, or by the point-switch method, illustrated in Fig. 110. Of course these figures are only diagrammatic and it should be at once understood that in these figures as well as in many others in this chapter, it has been necessary to use very short radii, very wide gauge, and very large frog angles in order to illustrate the principles by figures which are suitable for the page and which would at the same time be intelligible.

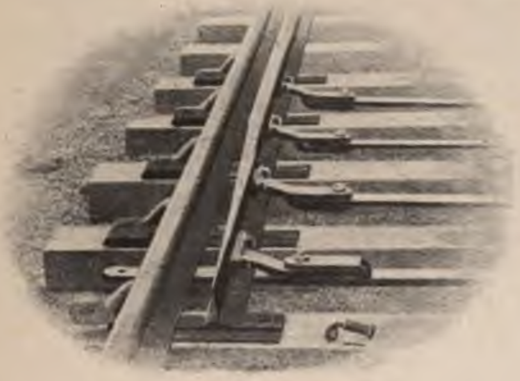


Fig. 111. Details of Point Switch.

The use of the stub switches is now confined to the cheapest of yard work or private switches which run off from sidings. They should never be used in any main track. Their construction may be implied from Fig. 109. The pair of movable rails are tied together at the proper gauge by tie rods. The two pairs of stub ends are of course fixed. The details of a point switch are illustrated in Fig. 111. Note that one rail on each side is absolutely unbroken. The other rail has nearly all of the head cut away and a part of one flange. The other flange and the web, with that part of the head immediately over the web still remains. The tie rods which are clearly shown connect this pared-down rail with a similar rail on the other side. The last tie rod has an extension to which the switch rod from the switch stand is attached. The moving rail slides on tie plates which have rail braces on the outer ends which stiffen the rail against the unusual lateral strain to which it is subjected. The angle of these switch points is generally 1° 50′.

Switch Stands. One type of switch stand, which also combines a semaphore (or signal which shows its position) is shown in Fig. 112. The mechanism is of course covered, the cover being indicated by the faint lines. The type shown is but one of a multitude for which there is no space here.

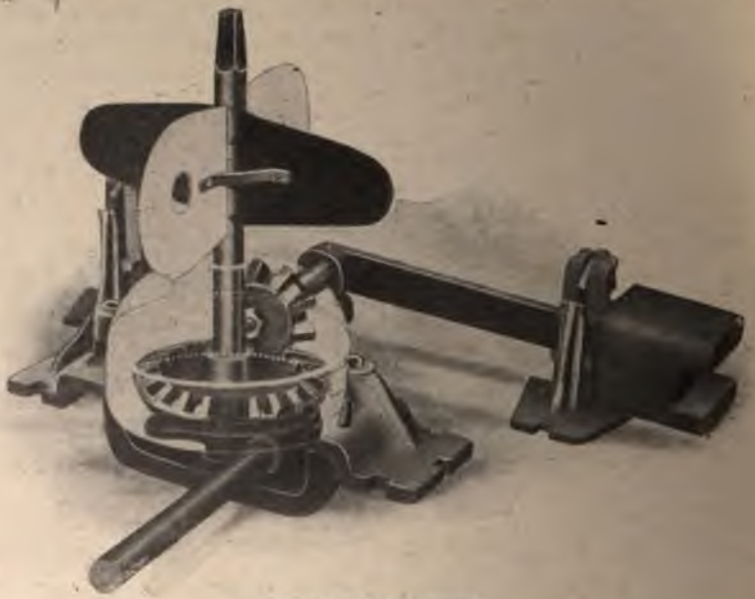
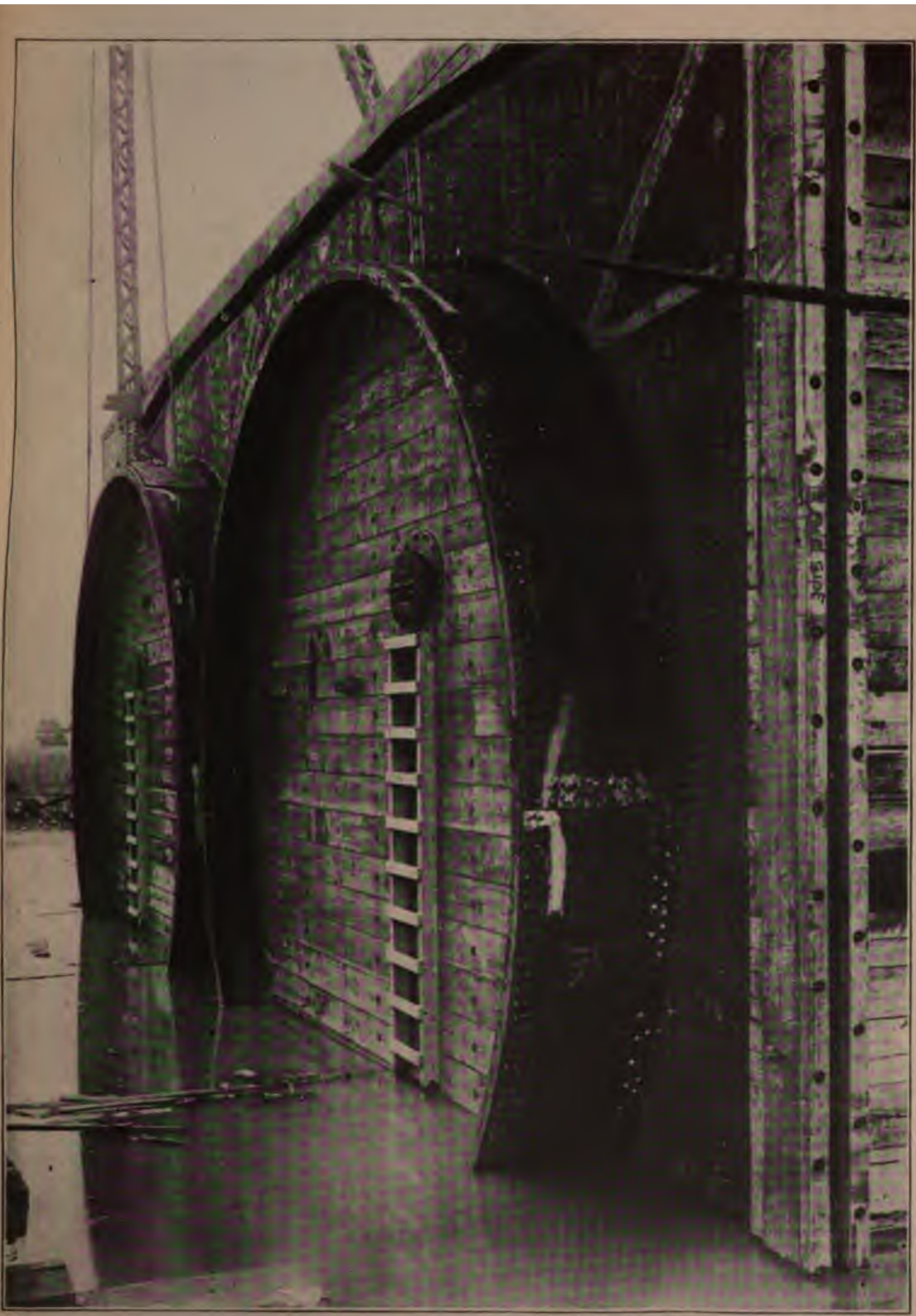


Fig. 112. Switch Stand.

Guard Rails. These are shown opposite the frogs in both Figs. 109 and 110. They obviate any danger of the wheel running on the wrong side of the frog point and also save the frog point from excessive wear. The flange-way space between the heads of the guard rail and the wheel rail must therefore not exceed a definite quantity, which is made about two inches. Since this is less than the distance between the heads of two ordinary sized rails when placed base to base, to say nothing of any space



END OF TUBE-SECTION FOR RAILROAD TUNNEL UNDER DETROIT RIVER

View showing bulkheads. Tubes to be sunk to desired level.

for spikes, the base of the guard rail must be cut away somewhat. These guard rails are made from 10 to 15 feet long and are bent a few feet from each end so that there shall be no danger that a wheel flange shall strike the ends.

Frogs. When the outer switch rail reaches the opposite main rail, the wheel flange must either pass *through* the head of the main rail or the wheel must be raised so that the flange may pass over the rail. The most commonly used frogs are those of the type of which the wheel flange passes through the head of the rail. The geometrical outline of such a frog is shown in Fig. 113.

The frog *number* may be found by dividing the distance from the "point" to any chosen place by the width of the frog at that place, or in the figure *ch* ÷ *ab*. But since *c* is the imaginary intersection of the sides produced and is not easily determinable with accuracy on the frog, it is sometimes easier to measure the width at two places (*ed* and *ab*) and then divide the sum of those widths by the total distance *sh*; this will give the same result as before. This measuring may be done with any convenient unit of length such as a pencil or a spike. Find the place where the width of the frog just equals the unit of length and then step off that distance to the "point." The fundamental objection to all frogs of this type is that they make a break in the main rail which causes a jar when a train is run over the frog at high speed. If the frog is made "stiff" as is illustrated in Fig. 114, the track has the advantage of being literally *stiff*, but the wheels have to run over the gap. The design shown in the figure aims to obviate any drop of the wheel at any point and this will be fairly accomplished as long as the hardened steel faces can resist the wear which is very severe in the older and commoner designs.

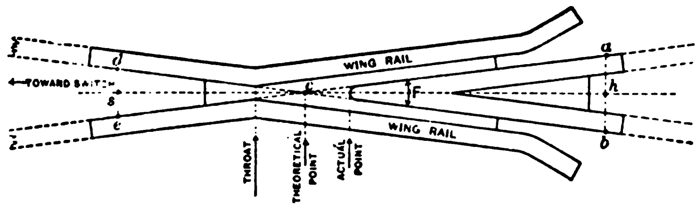
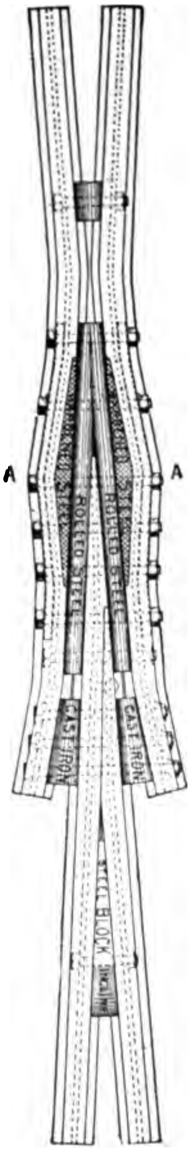


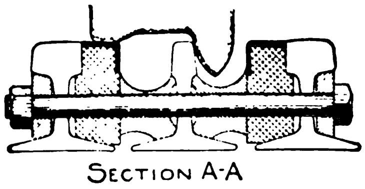
Fig. 113. Diagram of Frog.

The "spring-rail" frog, illustrated in Fig. 115, is an attempt to obviate the gap for main line trains. Wheel flanges running on to the switch force back a portion of the main track rail which is normally held in place by a heavy spring. Running on to the switch is supposed to be done at comparatively slow speed, which permits the rail to be forced back without danger of derailment. But since the main rail is kept in place by the pressure of a spring, the frog lacks the stiffness of a "stiff" frog. The method of raising the wheel and carrying it over the main rail is illustrated in Fig. 116, which shows one of the many devices to accomplish this end. The method has the very positive advantage of leaving the main track absolutely unbroken.

In Fig. 117 is shown a method of avoiding a break even at the switch. The switch rails are at the level of the main rails at the switch point but gradually rise higher until the wheel flange is high enough to cross over the main rail. Such a switch must be operated at slow speed.

120. Mathematical Design. In all of the following demonstrations, the track lines represent the gauge lines or the lines of the inside head of the rails. The older formulæ, which are still in extensive use on account of their simplicity, all assume that the switch rails are bent to arcs of simple curves extending from the switch point to the frog, and that they are tangent to the main rails at the switch point. On account of its common use and also because it forms a fitting introduction to the more





SECTION A-A

Fig. 114. Anvil-face Frog.

exact method, it will be given. In all of the following demonstrations, the following notation will, for simplicity, be kept uniform. R will represent the radius of curvature of the main track, if it is curved, and r is the radius of the switch rails. F will always represent the frog angle, and g the gauge of the track. L will represent the "lead" or the distance measured on the main track from the switch point B to the frog point F.

The angle FOD in Fig. 118 equals the angle F, and BD is the versed sine of F to the radius FO. From this relation we may derive the equation

$$r + \frac{1}{2}g = \frac{g}{\text{vers } F} \quad (58)$$

also, since BF ÷ BD = $\cot \frac{1}{2}F$, BD = g and BF = L, we have

$$L = g \cot \frac{1}{2}F \quad (59)$$

Also,

$$L = \left(r + \frac{1}{2}g\right) \sin F \quad (60)$$

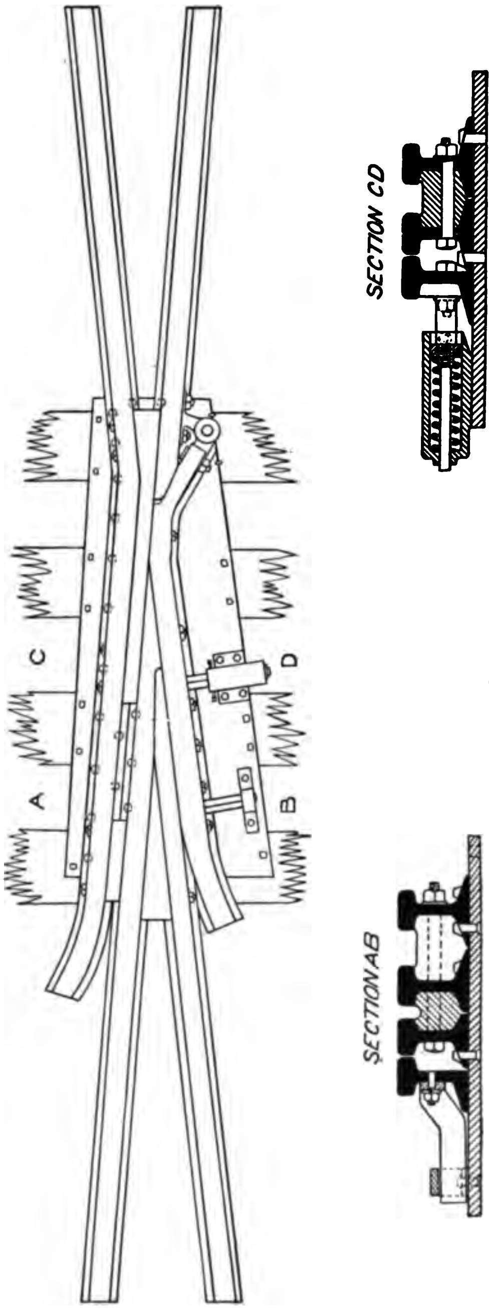


Fig. 115. Spring-rail Frog.

and $$QT = 2r \sin \frac{1}{2} F \qquad \textbf{(61)}$$

All of the above formulæ involve the angle F. Reference to Table III will show that with one chance exception the values of F are always odd and the accurate computation of their trigonometrical functions is tedious. Fig. 119 shows that the ratio of the length to width of a frog, or $pe \div ab$, which is called n, is also equal to $\frac{1}{2} \cot \frac{1}{2} F$. This relation can be used to derive the following marvellously simple formulæ:

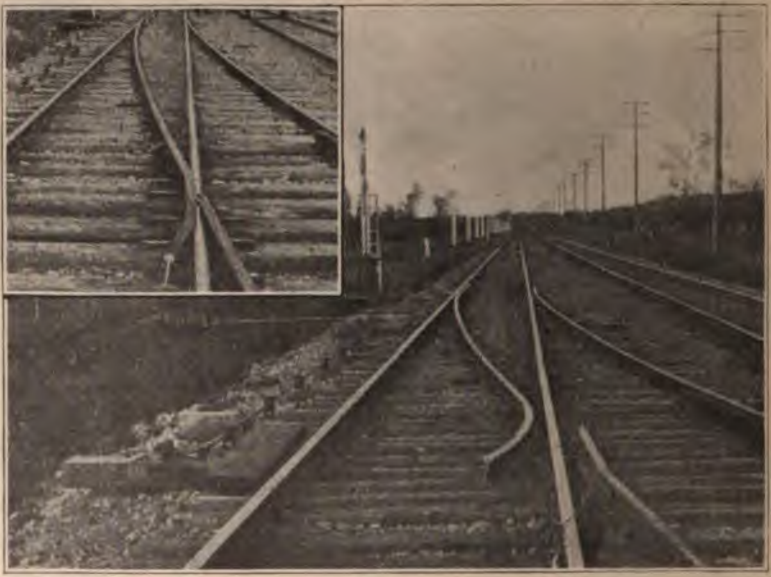


Fig. 116.

Since $L = g \cot \frac{1}{2} F$, and $n = \frac{1}{2} \cot \frac{1}{2} F$, we may at once derive the equation

$$L = 2gn \qquad \textbf{(62)}$$

But in Fig. 120 the line QZ, drawn midway between the rails, bisects DF at Z and also, since DQ is one-half of DB, QZ is one-half of BF or $= \frac{1}{2} L$. $OQ = r$ and the angle $ZOQ = \frac{1}{2} F$.

Then $r \div \frac{1}{2} \mathrm{L} = \cot \frac{1}{2} \mathrm{F}$, from which

$$r = n\mathrm{L} \qquad (63)$$

Combining equations 61 and 62, we have

$$r = 2gn^2 \qquad (64)$$

The above relations only lack the merit of correctness of application to make the whole subject very simple. They were first devised when stub switches were in universal use and although

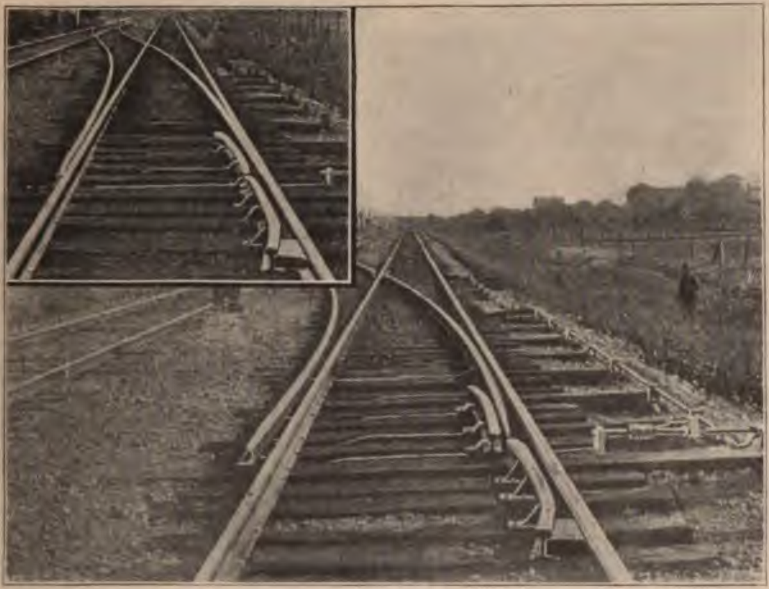


Fig. 117.

it is theoretically possible to make a stub switch conform to these lines, it is impracticable even there. But with point switches, which are in almost universal use, the switch rail makes an angle which is usually 1° 50′ with the main rail. The frog rails are also made straight.

The effect of each of these changes, taken separately, is to shorten the lead. The combined effect is to shorten the lead from 15 to 25 per cent. In Fig. 121, DM represents the straight point rail and HF the straight frog rail, the two being connected by the arc MH, tangent to both. The central angle of this arc is therefore $(F - a)$, a being the angle (MDN) of the point rail. The chord MH makes an angle with the main rails which equals

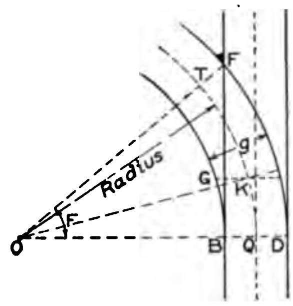


Fig. 118.

$$\frac{1}{2}(F - a) + a = \frac{1}{2}(F + a)$$

Call FH $= f$ and MN $= k$. Then HM sin $\frac{1}{2}(F + a) = g - f \sin F - k$. But HM $= (r + \frac{1}{2}g)\, 2 \sin \frac{1}{2}(F - a)$. Substituting this value of HM in the previous equation and solving for $(r + \frac{1}{2}g)$ we have

$$(r + \tfrac{1}{2}g) = \frac{g - f \sin F - k}{2 \sin \frac{1}{2}(F + a) \sin \frac{1}{2}(F - a)}$$

$$= \frac{g - f \sin F - k}{\cos a + \cos F} \qquad (65)$$

$$ST = 2r \sin \tfrac{1}{2}(F - a) \qquad (66)$$

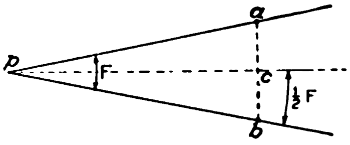


Fig. 119.

The lead BF $= L = HM \cos \frac{1}{2}(F + a) + f \cos F + DN$

$$= (g - f \sin F - k) \cot \tfrac{1}{2}(F + a) + f \cos F + DN \qquad (67)$$

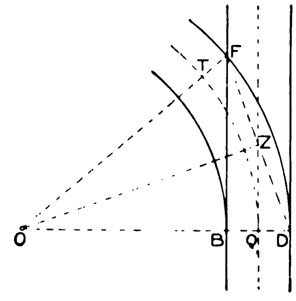


Fig. 120.

If $(r + \frac{1}{2}g)$ has already been computed numerically from equation 65, it will be more simple to compute L as follows:

$$L = 2(r + \tfrac{1}{2}g) \sin \tfrac{1}{2}(F - a) \cos \tfrac{1}{2}(F + a) + f \cos F + DN$$

$$= (r + \tfrac{1}{2}g)(\sin F - \sin a) + f \cos F + DN \qquad (68)$$

If the lead is computed for a turnout from a straight track using a No. 9 frog, a straight point rail and frog rail of the dimensions given in the middle section of Table III, it will be found that the lead becomes 72.20 instead of 84.75, the corresponding dimension assuming that the lead rails were circular throughout. Table III was computed on the basis of the above equations and the point switch dimensions which are in general use. The two references to section numbers in the table are to sections in Webb's "Railroad Construction," from which the tables were taken.

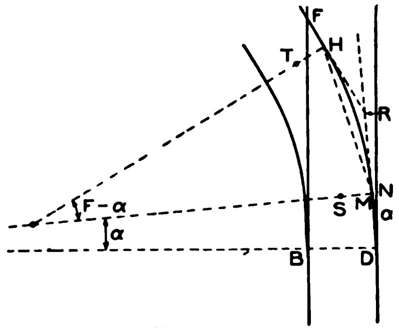


Fig. 121.

121. Turnout from the Outer Side of a Curved Track. When it is attempted to compute the dimensions of a turnout, from a curved track on the basis of using straight point rails and straight frog rails, it not only renders the demonstration exceedingly complicated, but it would involve assumptions regarding the mechanical construction which probably would not be followed in practice. Therefore the following demonstration is given with the purpose of showing the effect on the switch dimensions of curving the main track, the switch rails being circular throughout, and then drawing a reasonable inference as to the dimensions which should be followed for point switches from a curved main track. In the triangle FCD, in Fig. 122, we have

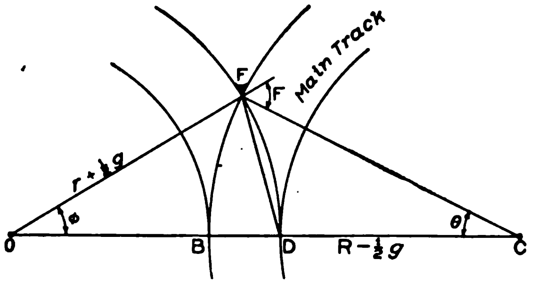


Fig. 122.

$(FC + CD) : (FC - CD) :: \tan\frac{1}{2}(FDC + DFC) : \tan\frac{1}{2}(FDC - DFC)$; but $\frac{1}{2}(FDC + DFC) = 90° - \frac{1}{2}\theta$, and $\frac{1}{2}(FDC - DFC) = \frac{1}{2}F$; also $FC + CD = 2R$ and $FC - CD = g$;

$$\therefore 2R : g :: \cot\frac{1}{2}\theta : \tan\frac{1}{2}F$$

$$:: \cot\frac{1}{2}F : \tan\frac{1}{2}\theta$$

$$\therefore \tan\frac{1}{2}\theta = \frac{gn}{R} \qquad \textbf{(69)}$$

Also, $OF : FC :: \sin\theta : \sin\phi$; but $\phi = (F - \theta)$

then $$r + \frac{1}{2}g = \left(R + \frac{1}{2}g\right)\frac{\sin\theta}{\sin(F - \theta)} \qquad \textbf{(70)}$$

The lead, $$BF = L = 2\left(R + \frac{1}{2}g\right)\sin\frac{1}{2}\theta \qquad \textbf{(71)}$$

A study of the three equations above will show that as the curvature of the main track increases and R grows less, $\tan\theta$ increases and θ increases. Then $(F - \theta)$ decreases and r increases. When $\theta = F$, as it readily may, $(F - \theta) = 0$ and r becomes infinity, that is, the switch rails become straight. If θ becomes greater than F, $\sin(F - \theta)$ becomes negative and r becomes negative. The interpretation of this is that the center of the switch track will be on the same side as the center of the main track. The figure will then correspond with Fig. 123 except that the positions of O and C and also of ϕ and θ will be transposed and also that "main track" should read "side track." Equations 73 and 75 will be the same as before, but equation 74 will be changed to

$$\left(r - \frac{1}{2}g\right) = \left(R + \frac{1}{2}g\right)\frac{\sin\theta}{\sin(\theta - F)} \qquad \textbf{(72)}$$

If we call d the degree of curve corresponding to the radius r, D the degree of curve corresponding to the radius R, and d' the degree of curve of a turnout from a straight track for the same frog angle F, it will be found that $d = d' - D$ very nearly. It will also be found that the "lead" as computed above and as computed for a straight track will agree to within a few inches and frequently to within a fraction of an inch.

Example. Compute from the above equations the values of L and *r* (and then of *d*) for the cases when the main track has a 4° degree curve and when it has a 10° curve; solve them for number 6, 9 and 12 frogs. This makes six cases. Compare them with values computed by the approximate rule.

In all these cases it may be shown that the discrepancies are very small. If such calculations are made for very sharp curves and for very large frog angles (which must be considered as bad practice), the discrepancies would be considerable, but since such turnouts (if ever made) should be operated at very slow speeds, the errors would have but little practical importance. Therefore we are justified in applying the approximate rule for turnouts from a curved track—use the same "lead" as for straight track; the degree of curvature for the switch rails to the *outside* of the main track will be the *difference* of the degree of curve for the main track and the tabular value for the degree of curve of the switch rails; for a turnout to the *inside* of a curved main track it may be similarly shown that the proper degree of curve for the switch rails is the *sum* of the degrees for the main track and the tabular value for the switch rails from a straight track.

Also, since it may be shown that the effect of using straight point rails and straight frog rails is to shorten the lead and to lessen the radius in approximately the same proportion, it may be assumed without material error that we may apply the same rule as above, and instead of taking the values of "lead" and "degree of curve" for the switch rails from the tabular form which uses circular switch rails throughout, we may take them from the revised form using straight switch rails and straight frog rails and apply the same rule.

122. Turnout from the Inner Side of a Curved Track. By the formation of precisely similar equations as were used in the previous section, we may derive the equation

$$\tan \frac{1}{2}\theta = \frac{gn}{R} \qquad (73)$$

From the triangle OFC we may derive

OF : FC :: sin θ : sin (F + θ), from which

$$\left(r + \frac{1}{2}g\right) = \left(R - \frac{1}{2}g\right)\frac{\sin\theta}{\sin(F+\theta)} \qquad (74)$$

$$\text{The lead BF} = L = 2\left(R - \frac{1}{2}g\right)\sin\frac{1}{2}\theta \qquad (75)$$

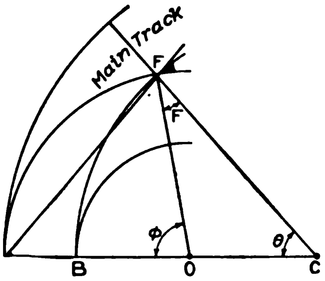


Fig. 123.

The details of the solution of the above equations should be worked out by the student; also a numerical demonstration of the fact, already referred to, that the degree of the turnout (d) is very nearly the *sum* of the degree of the main track (D) and the degree (d') of a turnout from a straight track when the frog angle is the same. It will be found that the discrepancy in these cases is somewhat larger than in the previous case, although it is still so small that it may be neglected when the curvature of the main track is small. An inspection of the figure will show that when the curvature of the main track is sharp the curvature of the turnout is very excessive. Such conditions should be avoided if possible, that is, a turnout should not be located on the inside of a very sharply curved main track if it can be avoided.

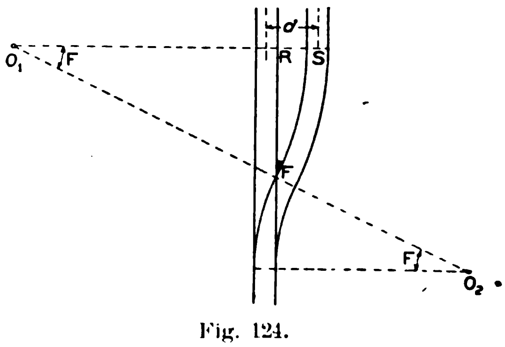


Fig. 124.

123. Numerical Examples. 1. Determine the lead and the radius of curvature for a turnout to the outside of a 4° 30′ curve using a No. 8 frog and a point switch.

2. Determine the lead and the radius of a curvature for a turnout to the inside of a 3° 40′ curve using a No. 7 frog and point switch.

In each of the above examples use the switch point angles, length of switch point and length of straight frog rails as given in Table III.

124. Connecting Curve from a Straight Track. The "connecting curve" is that part of the siding between the frog and the point where the siding becomes parallel with the main track, or the distance FS in Fig. 124. Call d the distance between track centers. The angle FO_1R must equal the angle F. If we call r' the radius of the connecting curve, we may say

$$\left(r' - \frac{1}{2}g\right) = \frac{d-g}{\text{vers } F} \tag{76}$$

$$FR = \left(r' - \frac{1}{2}g\right)\sin F \tag{77}$$

The distance FR may be shortened somewhat by the method indicated in Fig. 129. Theoretical accuracy would apparently require that we should consider a short length of straight track at the point F. The effect may readily be shown to shorten the radius r' and to shorten the distance FR by an amount exactly equal to the length of the straight frog rail, but in actual track laying such a procedure might be considered a useless refinement. And therefore in this case as well as in the succeeding similar cases, the effect of the straight frog rail will be ignored. It should likewise be noted that the figure has been drawn for simplicity as if the switch rails were circular. But since the point O_2 has no connection with the demonstration, it is immaterial what is the form of the switch rails back of F. This same remark applies to the following similar demonstrations.

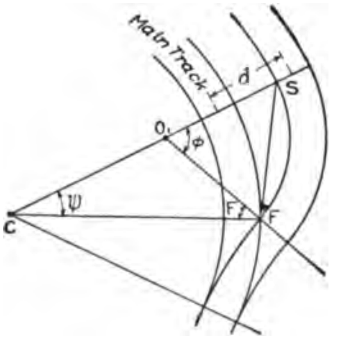


Fig. 125.

125. Connecting Curve from a Curved Track to the Outside. As in the previous case the only required quantities are the radius r of the connecting curve from F to S, Fig. 125, which must be determined from r and the angle ϕ ($= \mathrm{F} + \psi$). From the triangle CSF we may write

$$\mathrm{CS} + \mathrm{CF} : \mathrm{CS} - \mathrm{CF} :: \tan\frac{1}{2}(\mathrm{CFS} + \mathrm{CSF}) : \tan\frac{1}{2}(\mathrm{CFS} - \mathrm{CSF})$$

but $\frac{1}{2}(\mathrm{CFS} + \mathrm{CSF}) = 90° - \frac{1}{2}\psi$; and since the triangle $\mathrm{O_1SF}$ is isosceles, $\frac{1}{2}(\mathrm{CFS} - \mathrm{CSF}) = \frac{1}{2}\mathrm{F}$.

$$\therefore 2\mathrm{R} + d : d - g :: \cot\frac{1}{2}\psi : \tan\frac{1}{2}\mathrm{F}$$

$$:: \cot\frac{1}{2}\mathrm{F} : \tan\frac{1}{2}\psi$$ from which

$$\tan\frac{1}{2}\Psi = \frac{2u\,(d - g)}{2\mathrm{R} + d} \qquad (78)$$

From the triangle $\mathrm{CO_1F}$ we may derive

$$r - \frac{1}{2}g : \mathrm{R} + \frac{1}{2}g :: \sin\Psi : \sin(\mathrm{F} + \Psi)$$

$$\therefore r - \frac{1}{2}g = \left(\mathrm{R} + \frac{1}{2}g\right)\frac{\sin\Psi}{\sin(\mathrm{F} + \Psi)} \qquad (79)$$

Also $$\mathrm{FS} = 2\left(r - \frac{1}{2}g\right)\sin\frac{1}{2}(\mathrm{F} + \Psi) \qquad (80)$$

126. Connecting Curve from a Curved Track to the Inside.

There are three solutions according as F is greater than, equal to, or less than Ψ. In the first case, we may readily deduce, as in the previous section, from the triangle CFS (see Fig. 126) that

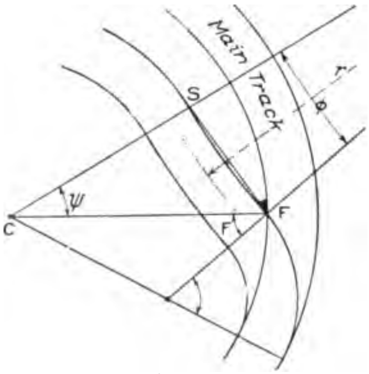


Fig. 126.

$$(2\mathrm{R} - d) : (d - g) :: \cot\frac{1}{2}\Psi : \tan\frac{1}{2}\mathrm{F}$$

and finally that

$$\tan\frac{1}{2}\Psi = \frac{2u\,(d - g)}{2\mathrm{R} - d} \qquad (81)$$

And as before, in equations 78 and 79, we may derive

$$\left(r - \frac{1}{2}g\right) = \left(R - \frac{1}{2}g\right)\frac{\sin\psi}{\sin(F - \psi)} \qquad (82)$$

and

$$FS = 2\left(r - \frac{1}{2}g\right)\sin\frac{1}{2}(F - \psi) \qquad (83)$$

When $\psi = F$, equation 80 will become

$$\tan\frac{1}{2}F = \frac{1}{2n} = \frac{2n(d-g)}{2R-d}$$ from which we may derive

$$2R - d = 4n^2(d - g) \qquad (84)$$

This equation gives the value of R which makes this condition possible. If we make $F = \Psi$ in equations 81 and 82, we find in the first case that r is infinite, which means that the track is straight, and in the second case that FS = infinity times zero, which is "indeterminate." But from the figure itself we may readily see that

$$FS = \left(R - \frac{1}{2}g\right)\sin\Psi \qquad (85)$$

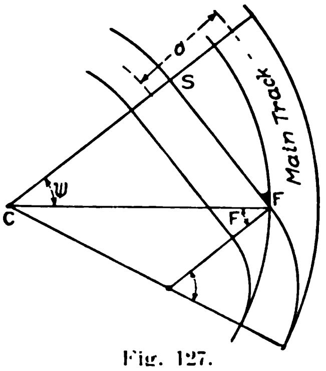


Fig. 127.

When $F < \Psi$ we may derive the value of $\tan\frac{1}{2}\Psi$ to be the the same algebraically as in equation 81, although the figure is so different. By the same method as before we may derive for the value of r the equation.

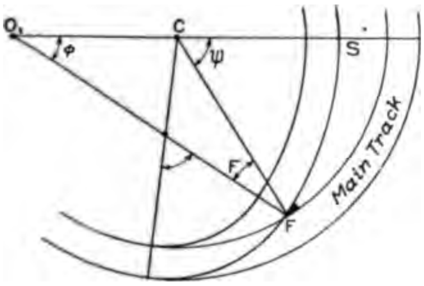


Fig. 128.

$$r + \frac{1}{2}g = \left(R + \frac{1}{2}g\right)\frac{\sin\Psi}{\sin(\Psi - F)} \qquad (86)$$

Also

$$FS = 2\left(r + \frac{1}{2}g\right)\sin\frac{1}{2}(\Psi - F) \qquad (87)$$

127. Crossover Between Two Parallel Straight Tracks. As in the previous cases, although the figures are drawn for simplicity with switch rails as simple curves, the demonstrations only involve the frog angles and the nature of the track beyond the frog. The better method is that shown by the full lines, when the track is straight between the frogs. But this consumes so much of the main track (many times what is indicated in the distorted figure) that a reversed curve (as is indicated by the dotted curves) may be used. The length of the straight crossover track is F_1T.

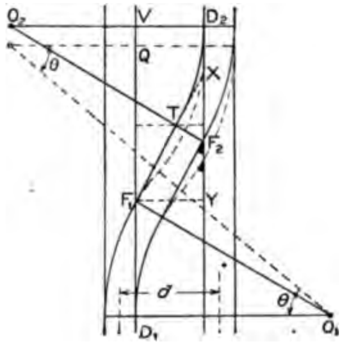


Fig. 129.

$$F_1T \sin F_1 + g \cos F_1 = d - g$$

$$F_1T = \frac{d - g}{\sin F_1} - g \cot F_1 \qquad (88)$$

The total distance along the track is

$$DV = D_1F_1 + YF_2 + F_2D_2 = D_1F_1 + XY - YF_2 + F_2D_2$$

but $\qquad XY = (d - g) \cot F_1$ and $XF_2 = g \div \sin F_2$

$$\therefore DV = D_1F_1 + (d - g) \cot F_1 - \frac{g}{\sin F_2} + D_2F_2 \qquad (89)$$

If a reversed curve with equal frogs is used, we will have the construction as is indicated by the dotted lines, and we have

$$\text{vers } \theta = \frac{d}{2r} \qquad (90)$$

also

$$DQ = 2r \sin \theta \qquad (91)$$

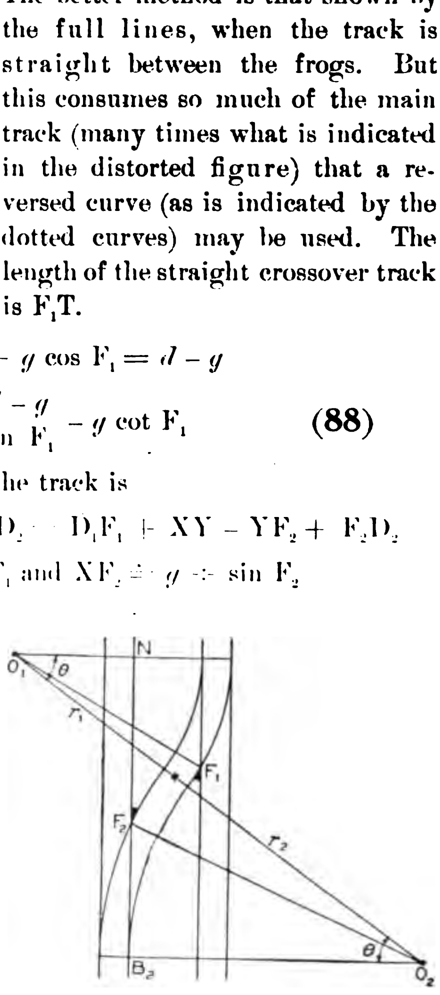


Fig. 130.

If it should for any reason be necessary to use frogs of different sizes, it may be done, but the point of reversed curve, instead of being in the exact center, will be as is indicated in Fig. 130. In this case we will have

$$r_2 \text{ vers } \theta + r_1 \text{ vers } \theta = d$$

$$\therefore \text{vers } \theta = \frac{d}{r_1 + r_2} \quad (92)$$

The distance along the track will depend, as before, on the length of the "lead" for each switch. If it were circular, as indicated in the figure, we would have

$$B_2N = (r_1 + r_2) \sin \theta \quad (93)$$

but the true lead for point switches would be less than this by the difference between the true L and $(r + \frac{1}{2} g) \sin F$. Therefore, this correction should be computed and subtracted for each switch.

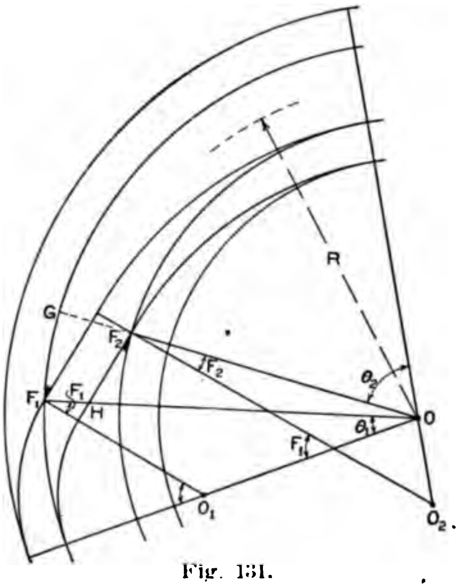


Fig. 131.

128. Crossover Between Two Parallel Curved Tracks. In the previous case there is no practical limitation as to frog numbers, but in this case there are limitations on what frogs are permissible. If the connecting track is straight, there are still three cases depending on the value of F_2, as in section 121. Two of these cases are illustrated in Figs. 131 and 132. The following

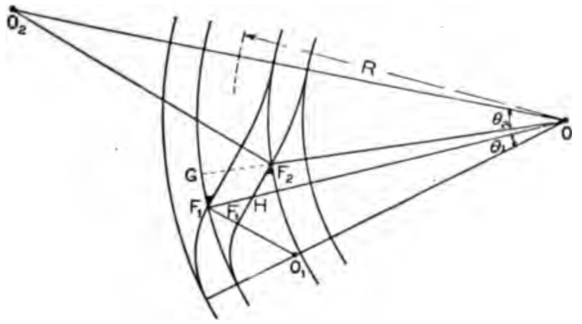


Fig. 132.

demonstrations apply to both figures. If one frog (F_1) is chosen, then F_2 becomes determined as a function of F_1. If F_1 is the angle for some even frog number, F_2 will in general be an angle that does not correspond to any even frog number and therefore will need to be made to order. If F_1 is less than some limit, depending on the width (d) between the parallel tracks, it will be impossible to have a straight connecting track, and at some other limitation it will be impossible to have the reversed curve connecting track shown later. In Figs. 131 and 132 assume F_1 as known. Then $F_1H = g \sec F_1$. In the triangle HOF_2 we have

$$\sin HF_2O : \sin F_2HO :: HO : F_2O$$

but $\sin F_2HO = \cos F_1$; $HF_2O = 90° + F_2$; $\sin HF_2O = \cos F_2$;

$$HO = R + \frac{1}{2}d - \frac{1}{2}g - g \sec F_1; \quad F_2O = R - \frac{1}{2}d + \frac{1}{2}g$$

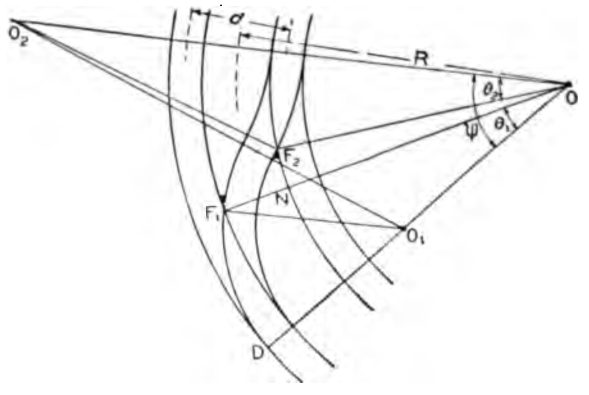


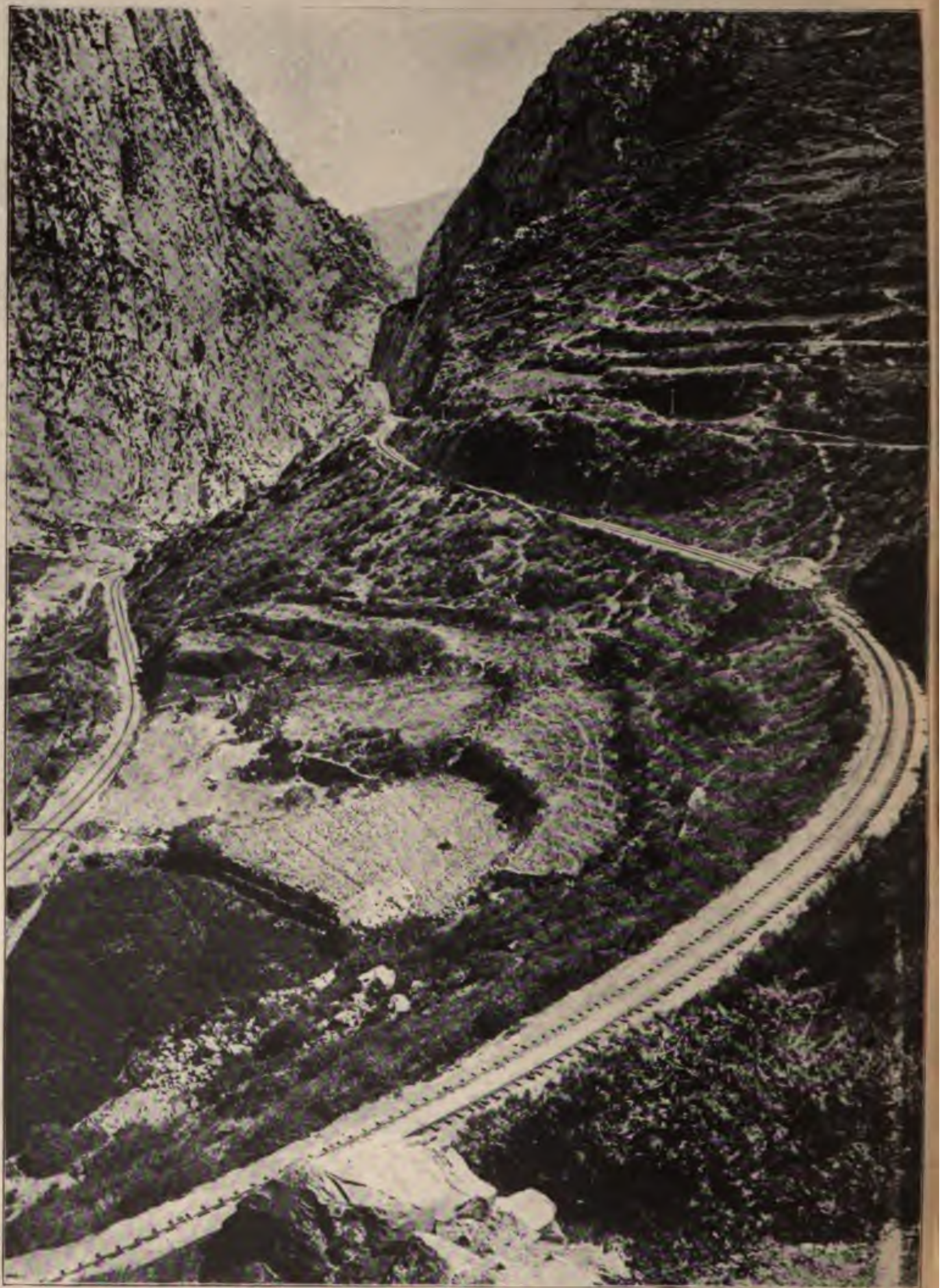
Fig. 133.

$$\therefore \cos F_2 = \cos F_1 \frac{R + \frac{1}{2}d - \frac{1}{2}g - g \sec F_1}{R - \frac{1}{2}d + \frac{1}{2}g} \qquad \textbf{(94)}$$

Knowing F_2, θ_2 is determinable from equation 69. To determine the relative position of the frogs F_1 and F_2,

$$HOF_2 = 180° - (90° - F_1) - (90° + F_2) = F_1 - F_2; \text{ then}$$

$$GF_1 = 2\left(R + \frac{1}{2}d - \frac{1}{2}g\right) \sin \frac{1}{2}(F_1 - F_2) \qquad \textbf{(95)}$$



RAILROAD BUILDING IN THE ANDES MOUNTAINS

Remarkable bend in the Cacray-Oroya line, Peru.

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If the connecting curve is made a reversed curve, as is shown in Fig. 133, the frogs F_1 and F_2 may be chosen at pleasure (within rather close limitations, however), and this will usually permit the adoption of regular standard sizes and will not necessitate the making to order of special sizes. We may then consider that F_1 and F_2 are known and that they are equal or unequal as desired. Employing formula 29 in Table XXX, we may write

$$\text{vers}\,\Psi = \frac{2\,(S - OO_2)\,(S - OO_1)}{(OO_2)\,(OO_1)}$$

in which $$S = \frac{1}{2}\,(OO_1 + OO_2 + O_1O_2)$$

but $$OO_1 = R + \frac{1}{2}\,d - r_1$$

$$OO_2 = R - \frac{1}{2}\,d + r_2$$

$$O_1O_2 = r_1 + r_2$$

$$\therefore S = \frac{1}{2}\,(2R + 2r_2) = R + r_2$$

$$S - OO_2 = R + r_2 - R + \frac{1}{2}\,d - r_2 = \frac{1}{2}\,d;$$

$$S - OO_1 = R + r_2 - R - \frac{1}{2}\,d + r_1 = r_1 + r_2 - \frac{1}{2}\,d;$$

$$\therefore \text{vers}\,\Psi = \frac{d\,(r_1 + r_2 - \frac{1}{2}\,d)}{(R - \frac{1}{2}\,d + r_2)(R + \frac{1}{2}\,d - r_1)} \qquad (96)$$

$$\sin OO_2O_1 = \sin\Psi\,\frac{OO_1}{O_1O_2} = \sin\,\frac{R + \frac{1}{2}\,d - r_1}{r_1 + r_2} \qquad (97)$$

$$O_2O_1D = \Psi + O_1O_2O \qquad (98)$$

$$NF_2 = 2\,(R - \frac{1}{2}\,d + \frac{1}{2}\,g)\,\sin\frac{1}{2}\,(\Psi - \theta_1 - \theta_2) \qquad (99)$$

The chief advantages of the above method are that it not only permits the use of standard size frogs, but also uses up less of the main track between the extreme switch points.

129. Problems in Switch Computation. 1. A siding runs off from a straight main track, using a No. 8.5 frog. The distance between track centers is 13 feet. What is the radius of the connecting curve and its length?

2. A siding using a No. 9 frog runs off from the outside of a 4° 30′ curve. What is the radius and length of the connecting curve? In all of these problems, consider the distance between track centers to be 13 feet.

3. Using the same frog, a siding is to run to the inside of the same track. What will be the radius and length of the connecting curve? Until Ψ is computed, it is impossible to say which of the three possible cases will be used, but the solution of equation 80 immediately decides that point, which will show that Ψ is slightly greater than F, but that the difference is so little that the resulting value r is very great. $\frac{1}{2}(\Psi - F)$ is such a small angle that Table VI must be used to determine its sine.

4. If a crossover is to be run between two straight parallel main tracks 13 feet between centers, using No. 8 frogs, how much will be saved in distance measured along the main track by using a reversed curve rather than a straight track? Since the *difference* in distance is called for, we may ignore in this solution the absolute length of the switch rails and consider that they would be the same in either case.

5. Required the dimensions for a cross-over between two main tracks which are on a 4° 30′ curve; the distance between track centers thirteen feet, the frog for the outer main track (F_1 in Fig. 129) is No. 9; F_2 is No. 7; the connecting curve is to be a reversed curve. When the radius of a double main track is given, it means the radius of the center line between the two tracks. We must, therefore (as indicated in Fig. 133), add and subtract 6.5 to the radius of a 4° 30′ curve (1273.6) to obtain the radii of the centers of the two main tracks. The figure and formulæ allow for this. Since point switches would unquestionably be used, we must determine r_1 and r_2 by the method outlined in §121; R_1 the radius of the outer main track = 1280.1 (which means that D_1 = 4° 29′), while R_2 the radius of the inner track = 1267.1 and D_2 = 4° 31′. Then by the rule of §121, r_1 = radius of $(d_1 + D_1)^\circ$ curve

= radius of (7° 31′ + 4° 29′) curve = 478.34; r_2 = radius of $(d_2 + D_2)$ curve = radius of (12° 26′ - 4° 31′) curve = 724.31. d_1 and d_2 are the degrees of curve given in the first section of Table III as being suitable for a No. 9 and a No. 7 frog on a straight track. Obtain θ_1 and θ_2 by substitution in equations 69 and 73. It will be found that the point of reversed curve comes but a fraction of an inch from the frog point F_2. If the computations had apparently indicated that the point of reversed curve would come beyond either frog point (or between either frog and its switch), it would have shown the impracticability of the use of a No. 7 and a No. 9 frog under these particular conditions. It shows that in this case the limit was practically reached.

6. Solve the same problem using a No. 9 frog in both cases. In this case it will be found that the total length of main track between the extreme switch points will be somewhat increased, but that the point of reversed curve will be nearly midway between the two tracks, as is preferable. A comparison of the two solutions will then show how close are the limitations in the choice of frogs to be used.

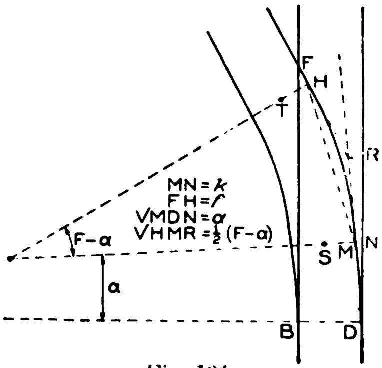


Fig. 134.

130. Practical Rules for Switch Laying. The following directions are based on the methods previously given for allowing for the effect of straight point rails and straight frog rails when used from a curved main track. When the position of the switch block is definitely determined, then there is no choice but to cut the main rails wherever the location calls for, but as the main track rail would be merely bent out to form the outer switch rail, there need be no rail cutting near the switch point, except that a rail-joint in the main rail should not come at or near the switch point. The frog has a length of from six to nine feet. A movement one way or the other of less than ten or twelve feet will bring one end of the frog at an existing joint and thus save one rail cutting.

After having definitely determined just where the switch is to be located, mark on the rails the points B, D and F in Fig. 134. Measure off the length of the switch rails DN, and locate the point M at the distance k from N. If the frog must be placed during the brief period between the running times of trains it will be easier to joint up to the frog a piece of rail at one or both ends of just such a length that they may be quickly substituted for an equal length of rail taken out of the track. When the frog is thus in place, the point H becomes located. The curve between M and H is a curve of known radius. Substituting in equation 54 the value of *chord* and R, we obtain x, or db in Fig. 135, which is the ordinate for the middle point of the curve. Then $a'' a$ and $c'' c$ will be three-fourths of db. Theoretically this will give a parabolic curve, but the difference will not be appreciable. Having located and spiked down the rail HM, the opposite rail may be easily put in at the proper gauge.

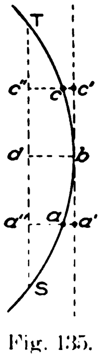


Fig. 135.

Example. *Locating a switch on a curved main track.* Given a main track having a 4° 30′ curve, to locate a turnout to the outside using a No. 8.5 frog; gauge, 4 ft. $8\frac{1}{2}$ in.; $f' = 3.19'$; $k = 5\frac{3}{4}$ in.; DN = 15 ft.; and $a = 1° 50'$. Then for a *straight* track r would = 600.94 ($d = 9° 33'$). For the curved track d should be nearly $(9° 33' - 4° 30') = 5° 03'$, or $r = 1134.9$. L for the straight track would be 69.60, but since the lead is slightly increased (say about 0.1—see § 121) we may call the lead 69.7, although this difference would be absolutely imperceptible after the track was laid, so far as train running was concerned. After locating the switch and frog point as described above, the frog and the switch rails should be placed. The chord ST is given in Table III as 51.38; the chord HM would be about 0.2 longer and curving the main track would make it slightly longer still. The best method is to measure it directly, but it should measure about 51.6. R = 1137.25. Applying equation 54, we have $x = 51.6^2 \div (8 \times 1137.25) = 0.293$, the ordinate at the middle point. The ordinate at the quarter-points is 0.219.

131. Slip Switches. The complicated demands for switching in yards and terminals have been greatly assisted by the device

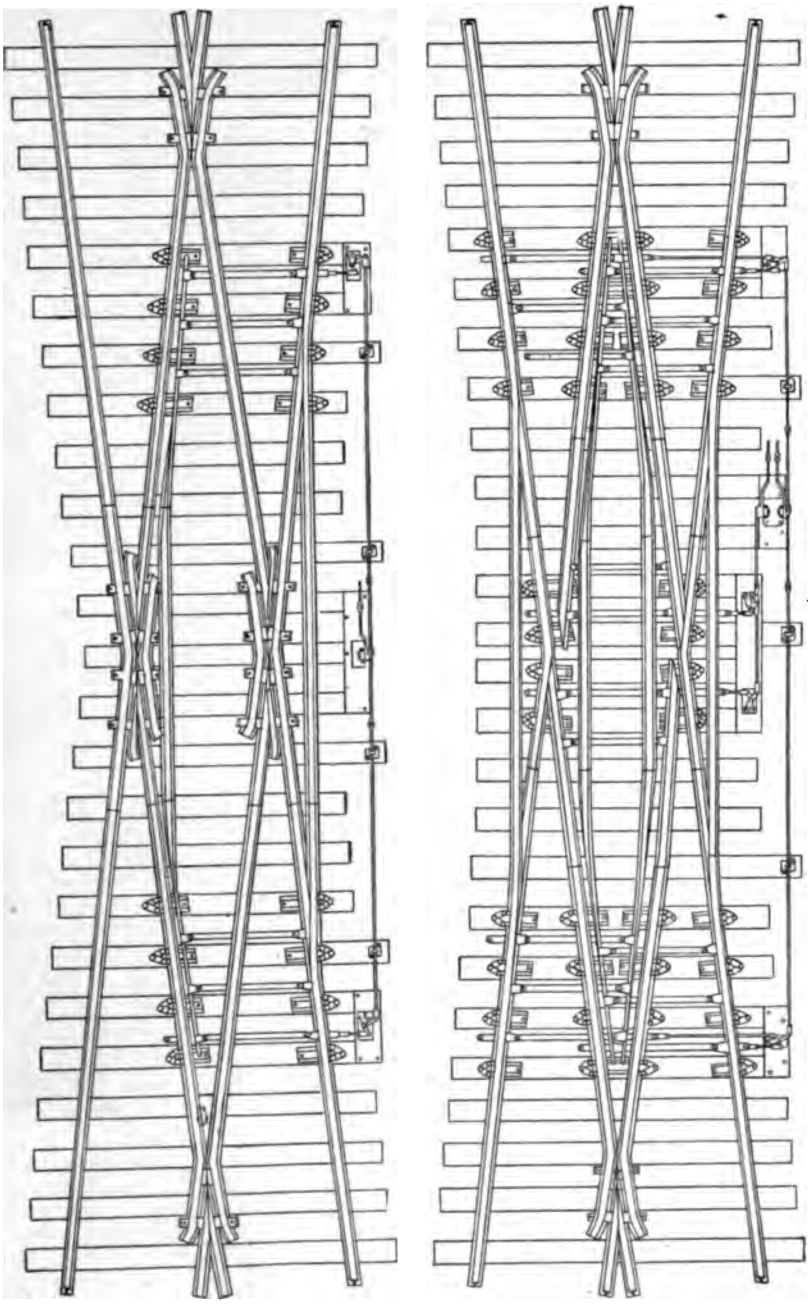


Fig. 136 Slip Switches. Fig. 137.

known as slip switches, illustrations of which are shown in Figs. 136 and 137. Fig. 136 shows a "single slip" in which the two middle frogs are fixed, although the system of movable frogs illus-

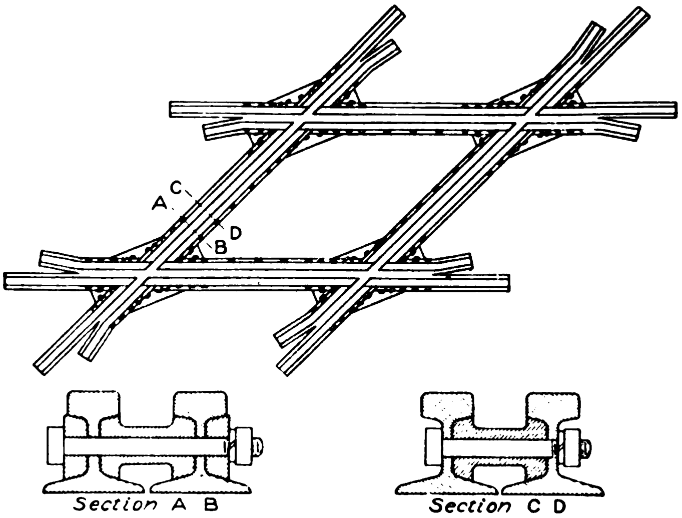


Fig. 138. Crossing.

trated in Fig. 137 is especially applicable. The double slip switch illustrated in Fig. 137 makes it possible for a train coming on either track to run directly on to either of the opposing lines. It should be noted that the mechanism is made interlocking so that the setting of a switch at one end will simultaneously set the switch at the other end as is required.

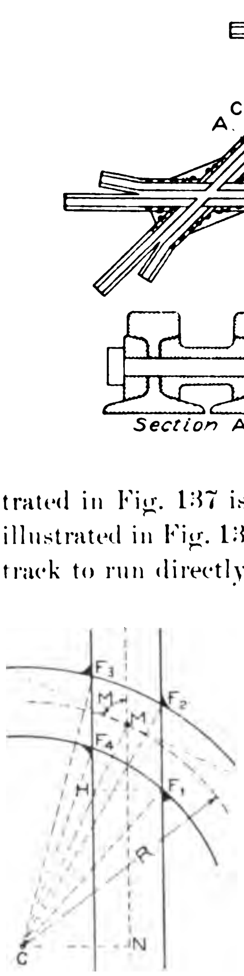


Fig. 139.

132. Crossings. When two railroads cross each other or even when it is desired to have one line cross another without having any switch connection, a crossing may be used. If the angle should be small (which is very undesirable) the method of movable frogs, shown by the crossing of the inner main rails of Fig. 137, may be used. But the lines should be required to cross each other as nearly at right angles as possible and then a bolted or riveted set of frogs, with fillers between the rails, such as is illustrated in Fig.

138, may be used. In general these crossings will need to be made to order according to the angle between the two lines. Since such crossings are sometimes operated at very high speeds the construction must be especially strong and rigid. When both tracks are straight the frog angles are identical, or more strictly, two of them are "complements" of the other two. When one or both tracks are curved, all four frogs will be different and the computation of their exact value becomes a somewhat complicated geometrical problem. The mechanical construction need not be essentially different from that shown in Fig. 138.

133. Crossing. *One straight and one curved track.* In Fig. 139, R is known and also the angle M, made by the center lines at their point of intersection.

$$\mathrm{M} = \mathrm{NCM} \text{ and } \mathrm{NC} = \mathrm{R} \cos \mathrm{M}$$

$$\text{then } \left(\mathrm{R} - \frac{1}{2}\,g\right) \cos \mathrm{F}_1 = \mathrm{NC} + \frac{1}{2}\,g$$

$$\left.\begin{aligned}
\therefore \cos \mathrm{F}_1 &= \frac{\mathrm{R} \cos \mathrm{M} + \frac{1}{2}\,g}{\mathrm{R} - \frac{1}{2}\,g} \\
&\text{Similarly it may be proved that} \\
\cos \mathrm{F}_2 &= \frac{\mathrm{R} \cos \mathrm{M} + \frac{1}{2}\,g}{\mathrm{R} + \frac{1}{2}\,g} \\
\cos \mathrm{F}_3 &= \frac{\mathrm{R} \cos \mathrm{M} - \frac{1}{2}\,g}{\mathrm{R} + \frac{1}{2}\,g} \\
\cos \mathrm{F}_4 &= \frac{\mathrm{R} \cos \mathrm{M} + \frac{1}{2}\,g}{\mathrm{R} - \frac{1}{2}\,g}
\end{aligned}\right\} \qquad (100)$$

To find the relative positions on the tracks of the frogs, we may write

$$\left.\begin{aligned} F_3F_4 &= \left(R + \frac{1}{2}g\right)\sin F_3 - \left(R - \frac{1}{2}g\right)\sin F_4 \\ HF_4 &= \left(R - \frac{1}{2}g\right)(\sin F_4 - \sin F_1) \\ F_1F_2 &= \left(R + \frac{1}{2}g\right)\sin F_2 - \left(R - \frac{1}{2}g\right)\sin F_1 \end{aligned}\right\}(101)$$

It should be noted that F_3F_4 will not be exactly equal to F_1F_2 although the difference will be very small.

134. Crossing. *Both tracks curved.* The angle of the tangents (or radii) at their point of intersection is a known quantity (M) and also the two radii R_1 and R_2. But since we must deal directly with the radii of the inner and outer rails of both curves, it will be easier to immediately add (or subtract) $\frac{1}{2}g$ to R_1 or R_2 and thus obtain r_1, r_2, r_3, and r_4, as indicated in Fig. 140. Applying formula 29 of Table XXX to the triangle $F_1C_1C_2$, and calling $s_1 = \frac{1}{2}(c + r_1 + r_4)$, we may write

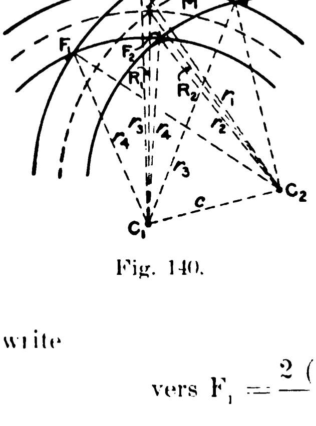


Fig. 140.

$$\text{vers } F_1 = \frac{2(s_1 - r_1)(s_1 - r_4)}{r_1r_4}$$

Similarly in the triangle $F_2C_1C_2$, let $s_2 = \frac{1}{2}(c + r_2 + r_4)$

and in the triangle $F_3C_1C_2$, let $s_3 = \frac{1}{2}(c + r_1 + r_3)$

and in the triangle $F_4C_1C_2$, let $s_4 = \frac{1}{2}(c + r_2 + r_3)$ (102)

and then we may write

$$\text{vers } F_2 = \frac{2(s_2 - r_2)(s_2 - r_4)}{r_2r_4}$$

$$\text{vers } F_3 = \frac{2(s_3 - r_1)(s_3 - r_3)}{r_1r_3}$$

$$\text{vers } F_4 = \frac{2(s_4 - r_2)(s_4 - r_3)}{r_2r_3}$$

To determine the length of track between the frogs we may write

$$\sin C_1C_2F_4 = \sin F_4 \frac{r_3}{c}$$

$$\text{and } \sin C_1C_2F_2 = \sin F_2 \frac{r_4}{c}$$

$$\therefore\ F_2C_2F_4 = C_1C_2F_4 - C_1C_2F_2 \qquad \textbf{(103)}$$

Knowing the angle $F_2C_2F_4$ we readily determine that the chord $F_2F_4 = 2r_2 \sin \frac{1}{2} (F_2C_2F_4)$. In a precisely similar manner the chords F_1F_2, F_1F_3, and F_3F_4 may be computed. As a check, it should be found that all these chords are nearly although not quite equal. Likewise the mean of all the four frog angles should be within a few seconds of the value of M.

135. Examples. 1. Determine the dimensions and the frog angles for the crossing of a straight track with a track of 4° curvature (as in Fig. 135) when the angle M = 72° 18′.

2. A 2° curve crosses a 4° curve as in Fig. 140, the angle M being 52° 20′. Determine the frog angles and the chord lengths between the frogs.

YARDS AND TERMINALS.

136. Value of a Proper Design. When a freight train arrives at a terminal yard, which is generally in a city of considerable size, with one or more other railroads or branches, the train load will in general be made up of some cars which will need to be shifted to some other road or division or to be shunted on to a siding where they may be unloaded. If the character of the train is mixed, partly coal and partly general merchandise or grain, the coal cars must be sent to their own tracks and the merchandise to theirs. A "division" point of a road is frequently the terminus of one or more branches as well as the point where freight trains are perhaps made up anew, especially if the ruling grade on adjacent divisions is so different that the train load which can be hauled by one engine is very different on the several divisions.

A little study of these facts, together with others which will readily suggest themselves in this connection, will show the vast amount of work which is necessary in sorting out the cars in a yard.

Often the road engine is cut off from the train as soon as it has brought it to its proper place in the yard, and the distributing is done entirely by switching engines. But the work in large yards is so great that several engines will be required for the work. The cost of running a switching engine per day may be figured as approximately $25. If the design of a yard can be so altered that one engine can be dispensed with, or that three engines may be made to do the work which formerly required four, we would have in 313 working days per year an annual saving of $7,825, which capitalized at 5%, gives $156,500 which is sufficient to reconstruct almost any yard.

As will be developed later, such a saving is by no means an impossibility. The requirements for space for water stations, ash-pits, coaling stations, turntables, sand and oil houses, engine houses, etc., and their proper arrangement so as to avoid useless running of the engines, is another feature which shows the value of a systematic design for a yard. When a yard is being constructed at a new place, it may be designed on the basis of subsequent work, no matter how little of it is immediately constructed, but very many yards were laid out when the now recognized principles were unknown. Subsequent additions have only made a bad matter worse until it is seen that an entire re-construction is necessary to make the yard what it should be.

137. Freight Yards. General Principles. A yard built on an ideal plan is in general an impossibility. Topographical considerations usually influence the problem to such an extent that the only method is to study the location so that certain fundamental principles may be applied.

1. A yard is a classifying machine for receiving, sorting and despatching cars to their several destinations as rapidly as possible. Its efficiency is measured by the rapidity with which it accomplishes this and the economy of motive power which is required.

2. At a yard which is the terminal of a division the freight trains are pulled in to a "receiving track" so as to get them out of the way and off of the main track. The road engine is then run off to the engine yard where it is cleared of ashes, loaded with water, coal, sand, etc., and otherwise prepared for its next trip. Perhaps the caboose is run off to a "caboose track" the location of

which is made convenient. Then, if the train is a "through" freight, another engine and caboose may be attached and it may proceed unbroken unless a change in ruling grade requires a different train load.

3. There are certain tracks in a yard which may be considered the skeleton of the yard. On these tracks no trains should be allowed to stand except temporarily. Such tracks, shown in Fig. 141, in which each pair of rails is indicated by a single line, are called "ladder tracks," and from these the storage tracks are run in parallel lines. Other through tracks are indicated on the plan.

4. The storage tracks should usually be made double-ended or with a ladder track at each end. This usually facilitates the switching by permitting one or more cars to be drawn from either end without disturbing the cars at the other end of that track.

5. In recent years many yards have been made by creating an artificial hump at such a place that the grade from the ladder tracks on to the storage tracks is about 0.5 per cent. This creates a gravity force of 10 pounds per ton which is sufficient to cause a car to roll by gravity from the ladder track on to any storage track to which it may be directed. In this way a train of cars on the ladder track may be distrib-

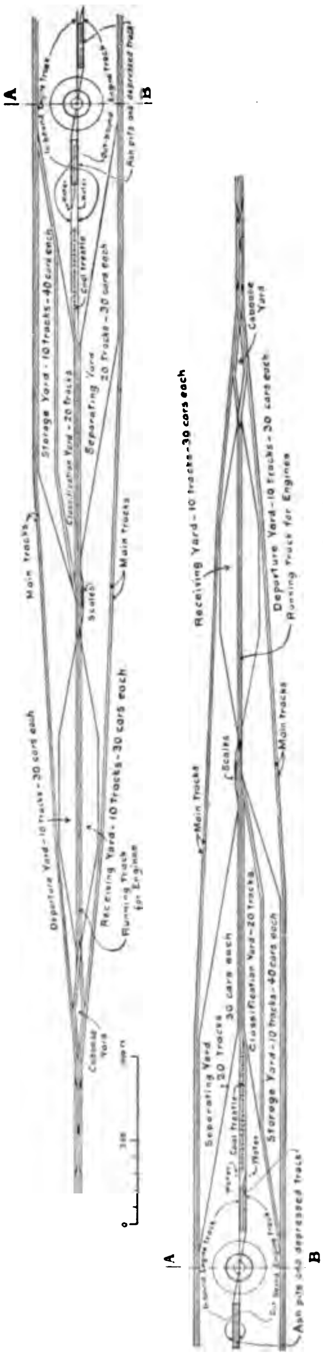

A
B
Storage Yard - 10 Tracks - 40 cars each
Classification Yard - 20 tracks
Separating Yard - 20 Tracks - 30 cars each
Coal trestle
Main tracks
Scales
Caboose Yard
Departure Yard - 10 tracks - 30 cars each
Receiving Yard - 10 tracks - 30 cars each
Running Track for Engines
Water
Ash pits and depressed track
A
B

Fig. 141. Plan of Typical Freight Yard.

uted to the various storage tracks with great rapidity. Incidentally there is no danger of a car running out from the storage track on to the ladder track. Symmetry and economy of space will usually require that the frogs and the switch dimensions of the switches running off from the ladder tracks shall be uniform. No. 7 frogs are very commonly used; frogs with a larger frog number make an easier riding track, but they require more space, and limit the space which may be used for storage. No. 6 and even No. 5 frogs are sometimes used on account of the economy of space which is thereby obtained, but it makes harder rolling and greater danger of derailment.

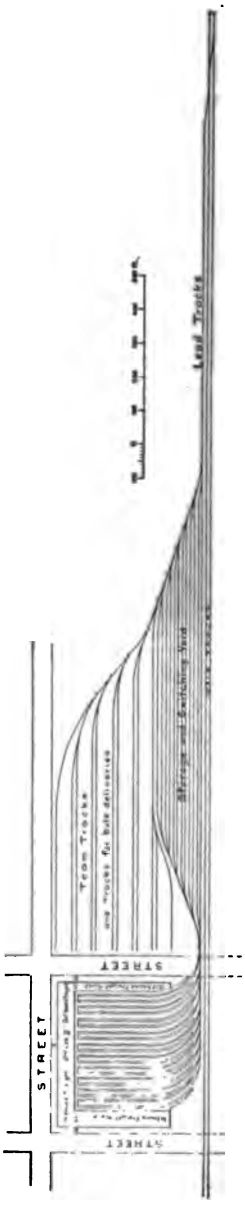


Fig. 142. Local Freight Yard.

138. Connection of Freight Yard with Main Tracks. As a general principle the main tracks should be as clear as possible from the yard tracks so that passenger trains may run through freely at any time without even the danger of a collision with any freight cars or of interfering with the work of the freight yard. This practically means that there should be no crossing of the main tracks by any tracks used in yard operations and that the connection should be only where it is desired to run from the main tracks on to the receiving tracks and that here the switches should be thoroughly protected by signals. The ideal construction is to have (on double track roads) all opposing tracks cross over or under each other so that two trains will never approach the same point of track except when they are moving in the same direction and then the danger of a collision will be largely averted. The receiving tracks (or similar tracks) should be utilized as "departing tracks" on which outgoing freight trains may wait for their signal to start without interfering with any passenger traffic on the main line tracks or any shifting work in the yard.

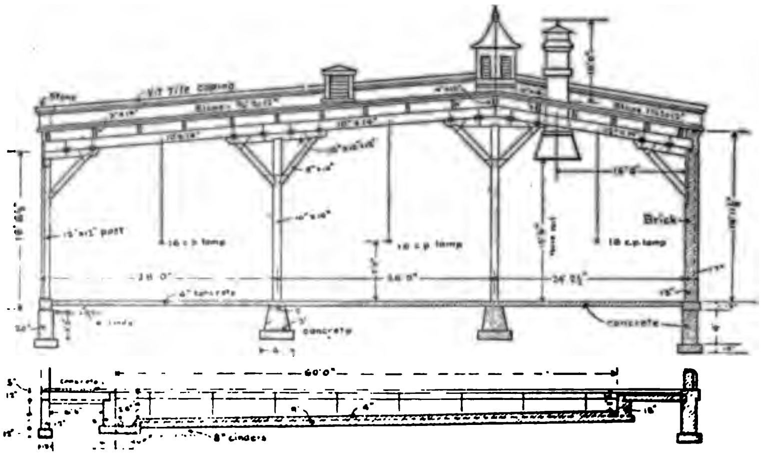
139. Minor Freight Yards. The name applies to the local collecting or distributing yards which are located in parts of a great city where the freight business is especially large. The cars are brought to these yards by means of long switches or by means of floats when the yard is located on a water front. The special feature of these yards is the fact that since they are always located on very valuable land, great ingenuity is required to utilize the limited space to the greatest advantage. This usually requires excessively sharp curvature, which may be limited by the fact that car couplers will not permit the car bodies to make an angle with each other much if any over 20°. This permits the use of radii as short as 100 feet, but this must be considered as about the limit. Radii as short as 50 feet have been used in some yards, but in that case an extension coupling bar is placed between the cars. Yards for receiving or distributing freight should be provided with team tracks which are made stub-ended and which are preferably placed in pairs with a sufficient space for roadway between each pair.

Figures 141 and 142 are ideal plans which were submitted to the American Railway Engineering and Maintenance of Way Association at its meeting in March, 1902. As "ideal" plans, it is not supposed that they can be literally adopted, but a study of them will show their general conformity with the principles stated above, and also will be suggestive of plans adapted to the local conditions.

140. Freight Yard Accessories. *Track scales.* These are for weighing freight cars on the track. When, as is frequently the case, the scales are located on a much used track, an auxiliary pair of rails is laid about six inches from the scale rails and connected with them by a split rail switch at a suitable distance from each end of the scales. One auxiliary rail is supported on the side of the scale pit and the other on several posts which run through the scale table floor. It has been found practicable to weigh a whole train load even in motion by running it very slowly over the scale tracks and noting the scale reading for each car when it becomes central over the pit.

Cranes. The frequent transportation of individual loads weighing many tons requires the use of some sort of unloader, which may vary from the temporary "gin pole" to a traveling crane which strides one or more tracks and a roadway, and which may travel on rails parallel with the switch tracks and also has a "traveler" which runs perpendicular to the tracks. The double horizontal motion (as well as the vertical motion) permits the loading or unloading between any car and wagon placed within its range. While their use is somewhat limited, there are occasions when they are almost indispensable.

141. Engine Yards. The ideal position for the engine house with its accessories is in the center of the yard, as is shown in Fig.



Longitudinal Section of Pit

Fig. 143.

141. The accessories of an engine house are shown in the ideal plan of Fig. 143. The plan of the cinder pit, which is shown in detail, allows for a pit about four feet deep under two tracks on which the engines run, and into which the ashes can be directly dumped. These tracks are each side of a depressed track which is sunk to such a depth that the sides of a gondola car will be below the bottom of the ashpits under the engine tracks. The dumped ashes can therefore be very easily shoveled into the car in the depressed track.

Passenger terminals for large cities are structures which demand the services of an architect rather than an engineer. The engineering features are largely those of elevating or depressing the approaching tracks so as to avoid the grade crossing of city streets, and all such problems must be solved individually. Those who wish to study the subject further may find it treated very fully in "Buildings and Structures of American Railroads," by Walter G. Berg.

SIGNALING.

The following description of signaling is not to be considered as a complete course on the subject as that would require more space than may here be devoted to it. The discussion has been condensed to such fundamental facts as every railroad engineer should know. The development of the science has been so rapid during late years that one must follow current engineering literature to keep abreast with the progress of the work. A student desiring a more throrough course in the groundwork of the subject is referred to "The Block System," by B. B. Adams (234 pages), as well as to similar but earlier works by W. H. Elliot and W. L. Derr.

142. Systems. When railroading was still in its infancy but traffic had so increased that rear-end collisions on double track became an imminent danger, two general plans were suggested and tried to guard against such accidents—(*a*) the time interval system and (*b*) the space interval system. Although some traces of the first system are still to be found in train order systems and in operating rules and time tables, it has been found inadequate for the operation of heavy traffic. When trains are run close together, even a short delay becomes a source of danger, which is only partially obviated by vigilant work by the rear flagman, and even this safeguard is only obtained at the expense of further delay in waiting for the flagman to return to the train after the cause of the delay is removed and the train is able to proceed. The space interval system has therefore become the basis of all modern systems.

Considered from another standpoint, the methods of handling trains may be divided into two general classes—(*a*) the telegraphic order system, in which men at different parts of the line receive orders by telegraph regarding the movements of trains which will soon pass them and who communicate these orders to the trainmen either verbally or by signal, and (*b*) those systems under which the signals at any point are controlled by mechanism at adjacent points. The fundamental difference between the two systems is that in the first case a blunder by any *one* of several men may cause an accident; in the second case, blunders are, to a considerable extent, mechanically impossible, and when made are generally immediately apparent to one or more others, and may be corrected in time to prevent an accident.

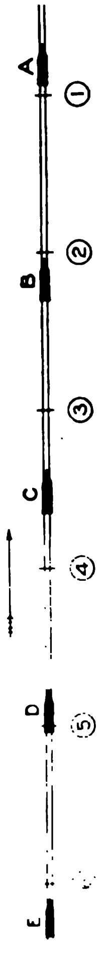
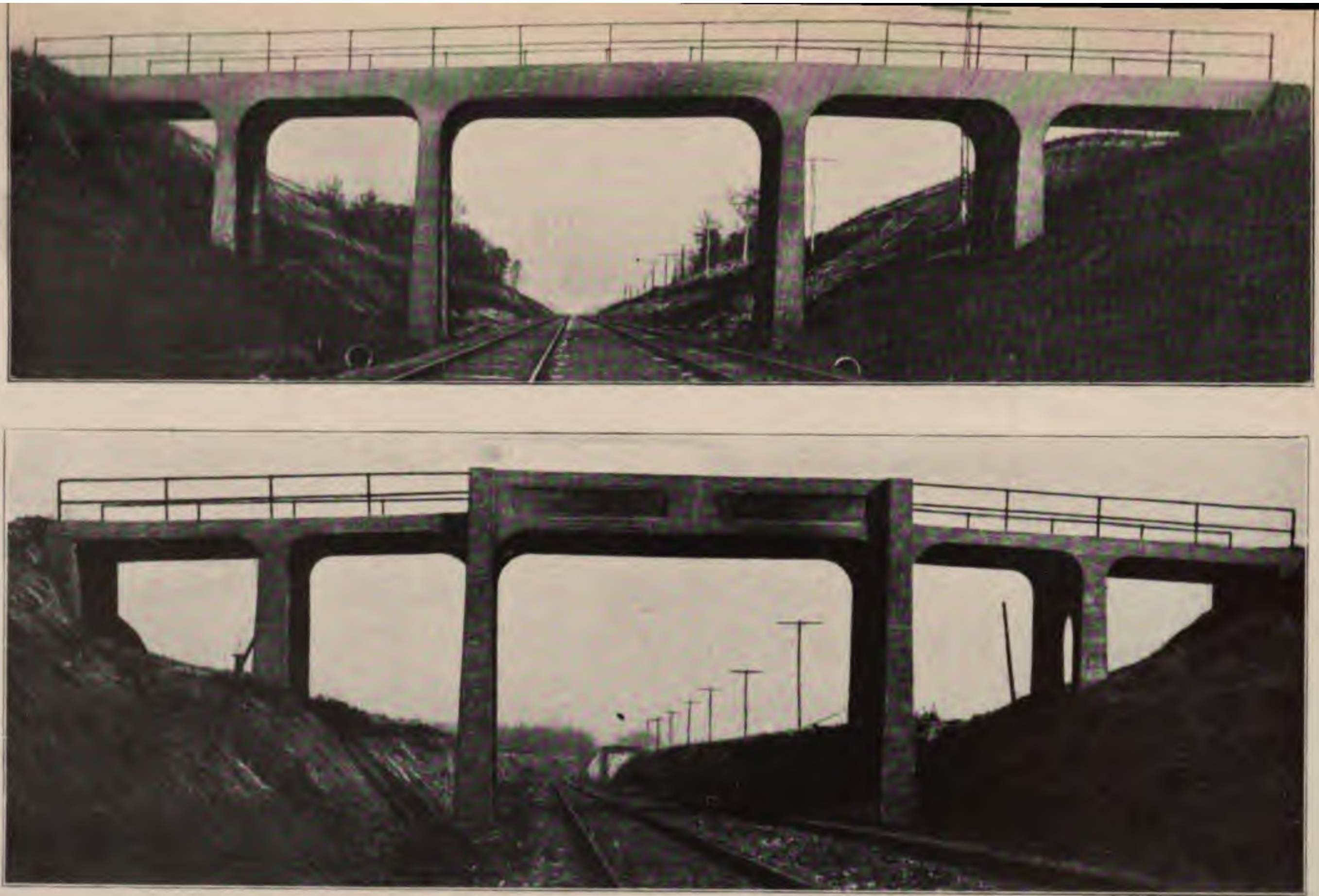


Fig. 144.

The first system includes the method by which a large proportion of the trains of the country are operated—the "train order" system, which will not be here elaborated since "signals" are not a necessary feature of it. Under this method the train crew receive their orders, issued by the train despatcher of the division, which are written out by the telegraph operator at the local office where received. The train is then run in accordance with such orders until it reaches the next train order office. The first system also includes the simple manual system, described in the next section. The various systems of controlling the signaling, culminating in the absolutely automatic system, will be successively described.

143. Simple Manual System. In this, as in all other block systems, the road is divided into sections or "blocks" whose lengths are varied somewhat to suit the method adopted and the natural conditions, and also are made roughly proportional to the traffic. For example, on the main line of the Pennsylvania Railroad between Philadelphia and Harrisburg the sections have an average length of a little over two miles, a few are four miles long, and some (especially where the suburban traffic is heaviest) are less than one mile long. On the other hand, on a road with less traffic (although sufficient to require the block system), the blocks might have a much greater length. "Absolute" blocking forbids the entrance of a train into



HIGHWAY CROSSINGS OVER THE BIG FOUR RAILWAY NEAR HILLSBORO, ILLINOIS

Built of reinforced concrete.

a block until the preceding train has passed out of it. This practically means that the trains must average considerably over one block apart, since train B (see Fig. 144) cannot enter the block (2 -1) until train A has passed out of that block, and the fact is telegraphed back so that the signals at (2) may be set for train B to enter the block. Train C and the succeeding trains must virtually maintain the same interval even though they temporarily move up closer. At a freight train speed of 15 miles per hour, trains could be run through blocks five miles long at intervals of twenty minutes plus the time required for signaling between stations and for the trains to pass by the signal station. Under the simple manual system the rules of operation, although varied in detail, are essentially as follows for double-track work:

When train A has passed (1) the operator there telegraphs the fact back to (2), and then the operator at (2) knows that the block from (2) to (1) is clear and that he can admit train B to the block. If train B does not arrive at (2) for some time afterward, (2) should obtain definite word from (1) immediately before B is due that the block is clear, since it might have become obstructed by switching operations or otherwise. As soon as train B has passed (2) the fact is telegraphed back to (3), which informs (3) that the block (3—2) is clear. The method of communication is usually by the ordinary Morse alphabet, but since the facts to be communicated are very few and simple, a system of taps on electric bells, which can be more easily and quickly learned than the Morse alphabet, are sometimes used. During recent years even the telephone has been used for this purpose. Some of the mechanical details of this method will be given later. Each road employing such a system has a more or less elaborate set of rules governing the operation of the signals, whose object is to make the work as mechanical as possible, to guard against giving wrong signals and to locate the blame when an error is made.

It should be noted, however, that there is nothing to prevent a signalman from giving a "clear" signal, when he should show a "stop" signal, even when he has been instructed otherwise and has perhaps reported by telegraph that he has obeyed orders. In short he is not "controlled," and in case of an accident there is a question of veracity between him and the engineman. The system has the merit of cheapness, since the signals may be of the cheapest form and the intercommunication may be done by the cheapest form of telegraphic circuit.

Permissive Blocking. There is a variation of the "absolute" system which is also applicable to some of the following systems and which facilitates traffic although at some sacrifice of safety. Under this system, a train is allowed to proceed into a block even though there is a train still there. But the train must be under "perfect control" (some rules limiting the speed to six miles per hour) so that it may be stopped very quickly if necessary. By this means, the delay of a succeeding train, and perhaps of several following trains, is very greatly reduced. Of course such a practice requires extreme caution to avoid accidents, and there are very minute rules to be followed when such running is permitted at all.

When heavy passenger trains are run at a speed approaching 60 miles per hour, it becomes impracticable to make a "service" stop much within 1,500 feet. Although a stop *may* be made in a much shorter distance, it induces very severe strains in the rolling stock and hence should be avoided. But since it is frequently impossible, on account of curves or other obstructions, to see signals more than a few hundred feet away, an engineman dare not approach a "home" signal at very high speed for fear a stalled train may be immediately beyond it. Therefore a "*distant signal,*" which forewarns the engineman of the indication of the "home" signal, is placed 800 to 2,500 feet from the home signal. The required distance, which for mechanical reasons is made as short as possible, except as noted below, depends on the grade and on how far from the signal it may be clearly seen.

When the distant signal is set for "clear," the engineman knows that he may proceed at least as far as the *second* home signal ahead; when it is set for "caution," he knows that he may proceed at least as far as the next home signal, but he must expect to be stopped there and he must have his train under such control that he can stop there if required. Sometimes the signal becomes cleared by the time he reaches the home signal and there is no actual delay beyond a slight reduction in speed, but the indication of the distant signal enables him in any case to approach the home signal confidently, knowing beforehand that it will be "clear" if the distant signal was "clear." In any system where the signaling is "controlled," such a distant signal is locked so that it cannot indicate clear when the home signal indicates stop. Under the "automatic" systems the distant signal is usually placed on the same post as the home signal for the preceding block. In this case, when the distant signal indicates clear, the engineman knows that his road is clear for two full blocks, but he may have to slacken speed when he reaches the next block station.

144. Controlled Manual System. In the previous system the only connection between the signal stations is the telegraphic communication of information. The "controlled manual" system includes the following essential elements. The signals at each station are locked by electromagnets which are controlled electrically from the signal station ahead. When a train approaches (*1*), (*1*) must notify (*2*) of it. If the last previous train has passed (*2*) and there is no other impediment, (*2*) will unlock (*1*)'s lever electrically, so that it is possible for (*1*) to set a clear signal. After the train has passed (*1*), the signal at (*1*) is set for the "stop" position. It will then be impossible for him to set it clear again until permitted to by (*2*). Knowing that the train is coming, (*2*) inquires of (*3*) if the block (*2*—*3*) is clear and if so (*3*) will unlock (*2*)'s lever so that it can be set for clear. The above is the simplest and earliest form of such a system.

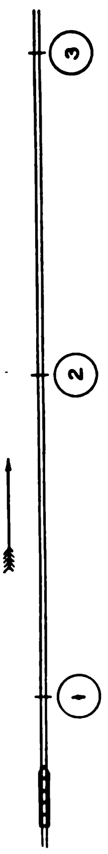


Fig. 145.

The chief advance over the simple manual system lies in the mutual control of the signal offices on each other. A signalman cannot set a signal clear except by the action of the next signalman ahead who thereby certifies that the block ahead is clear. The chances of error are thereby decreased. The electrical control is maintained over a "wire circuit," but the system is made much more under control by adopting features which are essentially those of the automatic system. The two rails of the track are carefully insulated from each other, and, near each signal station, the abutting rails are insulated at some joint by joining them with insulated joints such as are described in section 107.

At B, Fig. 146, a track battery sends a current through the rails which energizes the track relay at A, which operates the signal mechanism at A. The presence of even a single pair of wheels on the track between A and B, or even on the siding up to the "fouling point," will cause the current to be short-circuited and it will fail to energize the relay at A. By this means it is readily arranged that when the train passes A, A's signal will automatically fall to "stop" and will become locked there so that it cannot become unlocked until the train passes the insulated joints at B. When the train passes B, the current through the relay will then become strong enough to release the lock and then A can set his signal to "clear" if permitted to by B.

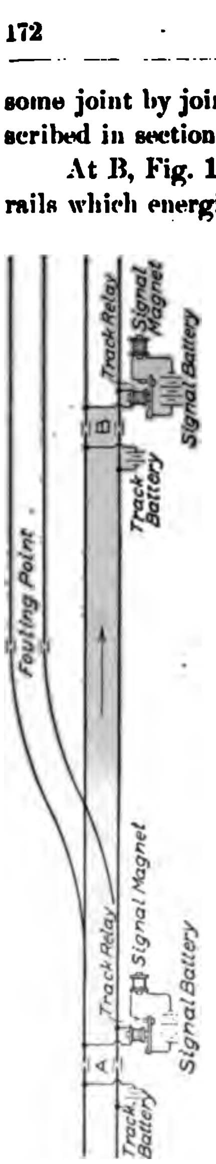


Fig. 146.

The method involves both a wire circuit and a track circuit. But when the sections are very long, it becomes very difficult to control the track circuit so as to avoid leakage and yet give the current sufficient strength to do its required work. And so the method is still further complicated by eliminating long stretches of the track circuit, but retaining it in the track near each signal station so that the signals will be automatically operated and controlled as before.

It should be noted that if a car was standing on the siding and was moved toward the switch point by wind, or through malicious mischief or otherwise, as soon as it passed the fouling point the signal at A would automatically go to "stop" and the signal would stay locked until the track was cleared. A broken rail would have the same effect of locking the signal and would start an investigation to determine the trouble.

145. Automatic Systems. Some of the principal essentials of the automatic systems have already been described above. Some of the differences are as follows. The mechanical work to be performed by the electric current in the controlled manual system is limited to unlocking certain mechanisms or unlocking the signals so that by gravity they will assume the "stop" position. The heavy work of moving the signals, which are usually of the "semaphore" type (described later) is performed by the signalmen. But automatic signals must be worked by a mechanism which always has sufficient power to move the signals. This practically means that the signals must have such a form and be so worked that but little force will be required to move them.

The earliest forms were targets mounted on a vertical axis which was swung around by clockwork. When set for "stop" a red target would show; when set for "clear" the red target would turn edgewise and a white target of different form which was previously edgewise (or perhaps no target at all) would then show. A lantern, with red lenses on two opposite faces and white (or green) lenses on the other two faces would be set on top of the axis. A weight moving up and down in a hollow iron post, would be periodically wound up to provide the power. Each time the signal is changed from "stop" to "clear" or from "clear" to "stop" the axis turns one-quarter turn. One objection to the method lies in the fact that since putting even a handcar or a track gauge on the rails will turn the signal to danger and taking it off will restore it to clear, the mechanism will be made to work so often that it will require rewinding with annoying frequency and then perhaps become run down and fail to work.

To guard against one source of danger, the mechanism is made to open the circuit and thus put the signal to "stop" just before it becomes run down, so as to avoid the possibility of the signal indicating clear when it should indicate danger. The clockwork system is still in successful use on some of the systems where it has been installed many years, but the more recent designs use an enclosed disk signal (described later). An important detail is the placing of the signal 200 feet in advance of the entrance of a block section. This enables the engineer to *see* the signals turn to danger as a result of his entering the block and he thus knows that there is a signal protecting him until he reaches the next signal.

If the signal fails to work, it shows that there is something wrong with the mechanism and he will take precautions accordingly.

Another advantage of the track circuit system lies in the fact that if a switch be opened anywhere in a block, the switch being provided with a circuit breaker, the circuit will be broken and the signal will automatically fall to danger. In short, almost any defect or impediment to a clear track will be indicated by the signal. And herein lies one troublesome feature: the circuit is so sensitive that any accidental short-circuiting (even though not due to any defect or obstruction of the track) will delay traffic. The opposite (and far more serious) error in operation—indicating "clear" when it should indicate "stop"—will only be caused by a defect in the mechanism, and the record in that respect is very good, the proportion of such errors to number of signal movements being exceedingly small.

146. Mechanical Details. The train order system does not necessitate signals of any kind, but on many roads which make no claim to a block signal system a signal of some sort will be displayed from the local train-order office. The signal may be a mere flag on a stick; an improvement is to hang it from a horizontal support, the lower edge being weighted, the whole being provided with a cord which is run back to the office, which permits the ready display or removal of the flag. Some western railroads have improved these by using some "home-made" signals operated similarly, but using a target made of thin wood or of sheet metal. From this it is but a short step to the standard "semaphore," illustrated in Fig. 147 and elsewhere.

The semaphore consists essentially of a board about five feet long, eight inches wide at the outer end and six inches wide at the hinge end. The hinge is a somewhat elaborate casting with one or more "spectacles" as holders of colored glass lenses. Since the weight of the casting on the spectacle side is usually not sufficient to overbalance the weight of the semaphore board, a counterweight is so attached that if the rods or wires to the signal cabin should break, the signal will automatically assume the horizontal position, which is universally considered as the "stop" or "danger" signal. When the axis of the board passes through the hinge bolt, as is shown in Fig. 147, the "clear" position is given by inclining the board at an angle of 45°, as shown in position B.

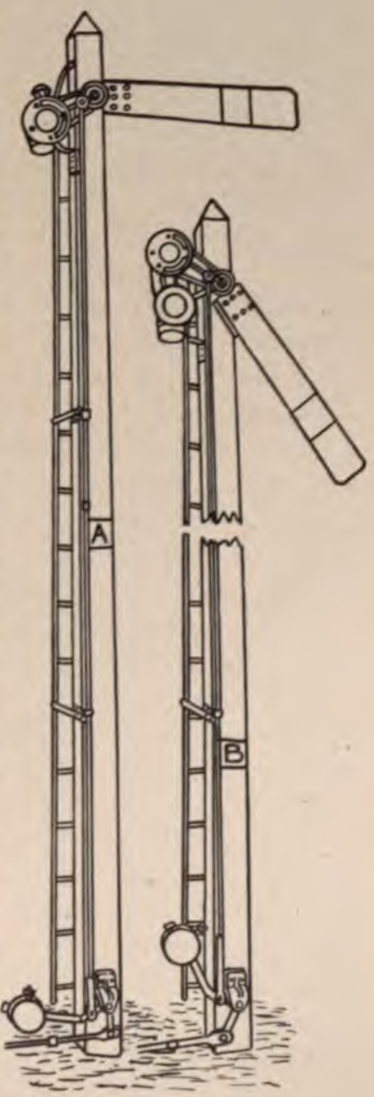


Fig. 147. Semaphore.

Another form is to have the board eccentric to the hinge, so that it may be dropped to a vertical position and still show outside of the post. As a general principle of construction, the board should be clearly visible even in foggy weather, and therefore the board should not come down directly in front of the post, for in foggy weather it would not be clearly visible and an engineman might pass the signal thinking it was in front of the post, when it might have been broken off and should have indicated danger. Fig. 147 shows a wooden post; the latest high-grade practice now uses iron posts with suitable castings at top and bottom. One advantage of such posts is the placing of the rods inside of the post where they are less subject to interference from snow and sleet and from malicious mischief.

The boards are always set so that they point to the right from the track which they govern, or in other words a signal which points to the left of its supporting pole, as seen by an approaching train, governs trains moving in the opposite direction. Sometimes the boards are painted red on the governing side and white on the other side, but whatever the variation in practice the indication is independent of the color, and on some roads the color is "neutral," so as to emphasize the fact that the engineman must be governed by the *form* and *position* of the board rather than by the color.

The only essential variation of form of the blade lies in making the ends of all home signals square and of all distant signals notched or of a "fishtail" form. One other form used for the distant signal is to make it pointed. When there are but two tracks the semaphores are usually placed on separate posts on each side of the roadbed. Even when there are four tracks, the signals for the two tracks on each side may be placed on one main pole which has a cross-arm and two uprights, each carrying one or more semaphores, as shown in Fig. 148. But when there are more than four tracks (as in yards), and frequently on four-track roads, the signals are carried on a "bridge" such as is illustrated in Fig. 149. In such a case the signals for each track are placed directly over the track.

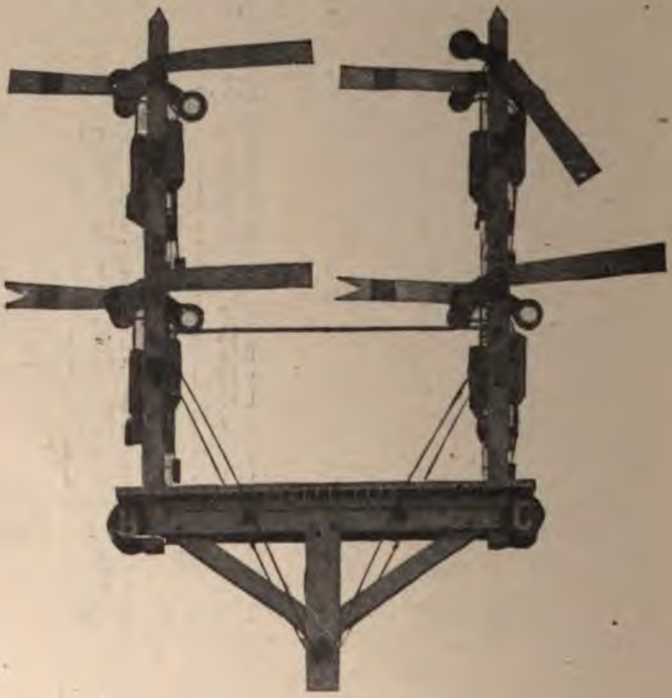


Fig. 148.

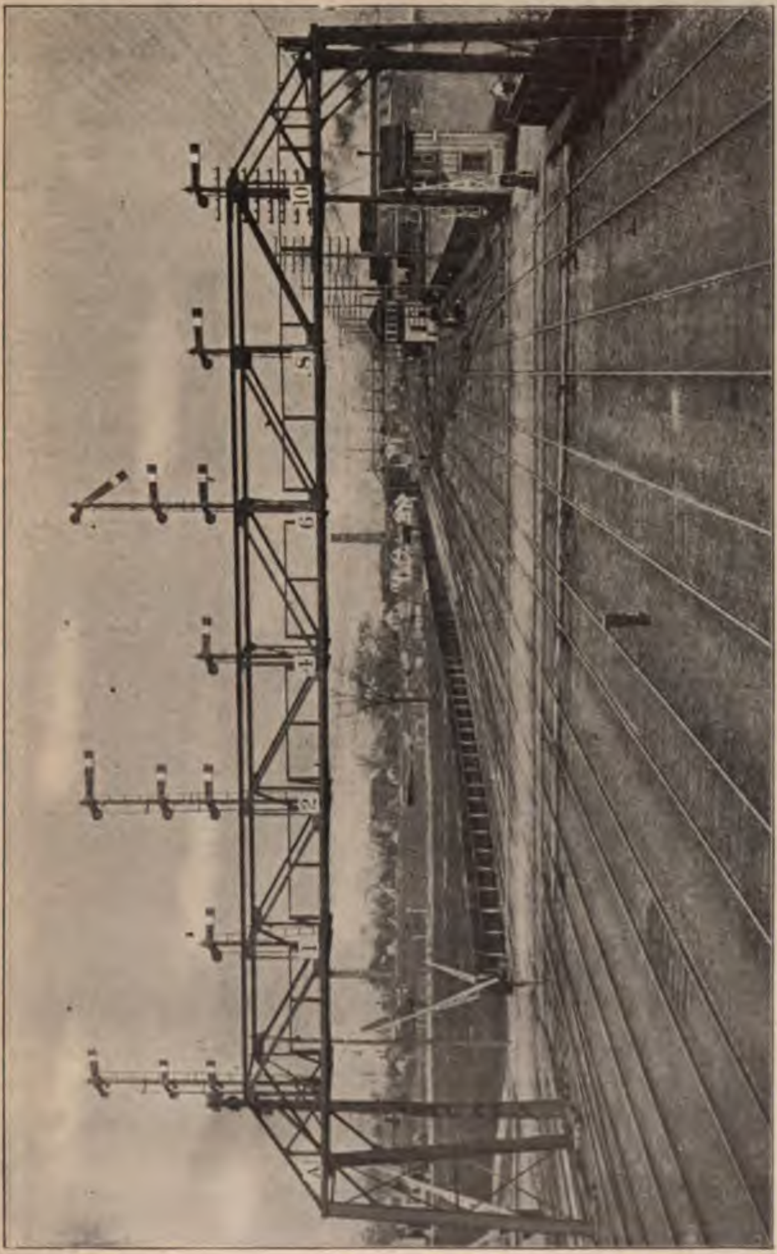


Fig. 149. Semaphore Bridge.

When more than one square-ended signal is over a track, the upper one refers to the through track and the lower ones to the switches which will be immediately encountered. Note in Fig. 149 that the signal bridge in the background has boards on the *left* side of the posts and that they are evidently *white*. This shows that the bridge governs movements *toward* the observer, while the signals on the bridge in the foreground evidently govern train movements in the direction the observer is looking. The mechanism of all such signals is necessarily somewhat exposed, and is liable to be actually blocked when covered with snow and sleet. A considerable amount of power must therefore be available to operate such signals.

Another form in extensive use is the enclosed signal.

Enclosed signals. There are two great arguments for and against the use of such signals. On the one hand, the mechanism is entirely enclosed and protected from the weather and is therefore uninfluenced by wind, snow or sleet. Also the mechanism can be made so very light and delicate that it requires only a small percentage of the power required to operate semaphores, and therefore they can be operated by an electric current of very low voltage. On the other hand, the signal is not one of *form* and *position* but of *color* only. It is argued that it cannot be as clearly seen in stormy weather and on that account is less safe. While it is unquestionably true that the signal indication is less visible in bad weather than a semaphore, yet the net advantages of the system are such that the system is very largely used.

The external appearance of the top of the signal (the post being omitted in the illustration) is as shown in Fig. 150. "Clear" is indicated by the disk opening showing white. To indicate danger a very light screen, made by stretching red silk over a light hoop, is swung over the opening. At night the lantern on the rear side shines through the opening, showing white or red according to the position of the screen. The detail of the mechanism, shown in Fig. 151, explains its operation. When the magnet is energized the disk is drawn up out of view and the signal shows white. If the current fails for any reason, the disk falls by gravity and comes into view. The power required is so small that the magnet not only controls the signal but also develops the power to move it.

147. Wires and Pipes. Wires are used for the transmission of electric current and pipes are used to transmit pneumatic pressure—as discussed later. But the above heading refers to wires

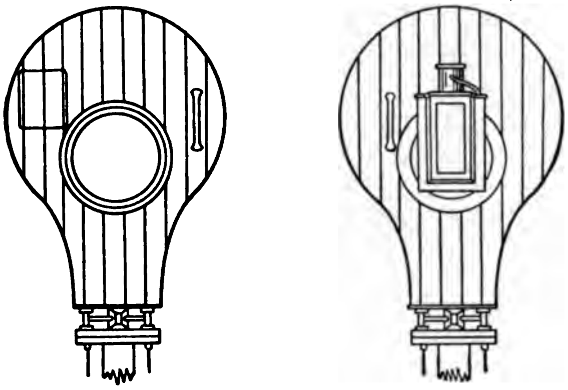
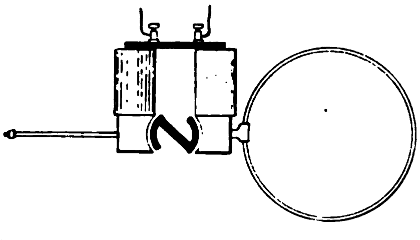


Fig. 150. Enclosed Signal.

and pipes as used to mechanically transmit motion from the signal cabin to the signal. When the parts may be made to work by tension, No. 9 wires may be used. When it is required to turn a right angle a grooved wheel is used and a short length of chain



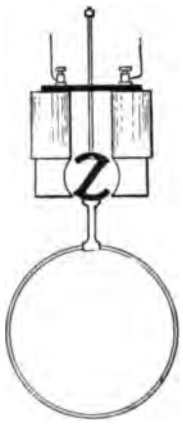


Fig. 151. Hall Automatic Signal Magnet.

is substituted for the wire. Slight deflection for pipes is accomplished by means of a series of bent rods running through guides, as shown in Fig. 152. If the deflection is greater, each rod must have a "bell crank." It is possible to work a signal with one wire, depending on gravity for the reverse motion, but good practice requires a wire for each motion. Signals are sometimes operated mechanically at a distance of 2,000 feet from the cabin. For such, wires are practically a necessity, but when the signals are nearer, pipes which may exert a push as well as a pull are used.

Compensators. The coefficient of expansion of iron is so high that the change of length in a wire or pipe several hundred feet long is so great that the signaling mechanism is thrown out of adjustment unless there is some automatic device to counteract it. The change of length of 1,500 feet of wire due to a fall of temperature from 100° F. to 20° is 1500 × 80 × .0000065 = 0.78 foot = 9.36 inches. A much less change than this would require adjustment. The geometrical principle of the automatic compensators is shown in the upper part of Fig. 153 and the practical construction is shown below it. By reference to the figure it may be seen that if the pipe ab contracts so that b moves to b', the point c would be moved to c', where $bb' = cc'$. But if $cd = ab$, dc would also contract to dc'. Therefore if the compensator is placed midway between the cabin and the signal, the cabin end of the pipe being fixed, a point at the signal end would retain its position regardless of any temperature change.

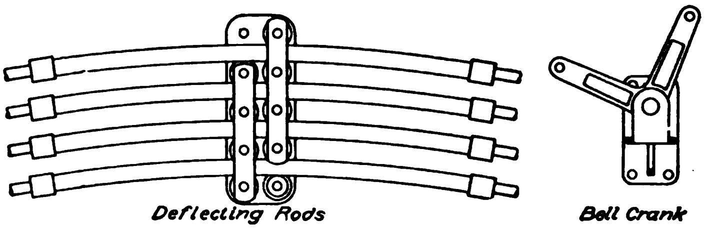


Fig. 152.

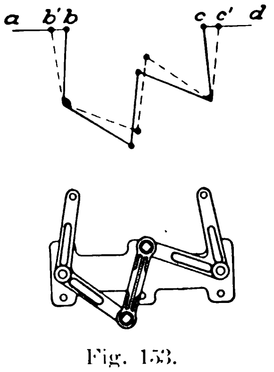


Fig. 153.

Practically these arcs should not be required to work through too great an angle. It has been found that 500 feet is a desirable limit. Therefore if a signal was 1,000 feet away from the cabin, two compensators should be used, each placed 250 feet from the ends. Then the position of the ends and the middle point would be unchanged by temperature. It should be noted that the insertion of such a mechanism changes the direction of the motion of the pipe; *i.e.*, if *ab* moves to the right *cd* will move to the left, and *vice versa*. Therefore one section or the other must be in compression, and such a compensator is applicable only to pipes. No compensator which is equally satisfactory has ever been designed for use with wires. They all require a spring or weight which takes up the slack, but if the wire gets caught somewhere this spring or weight may be pulled because its resistance is less, and then the signal does not operate. Several designs are in use and they work satisfactorily as long as the mechanism is in order.

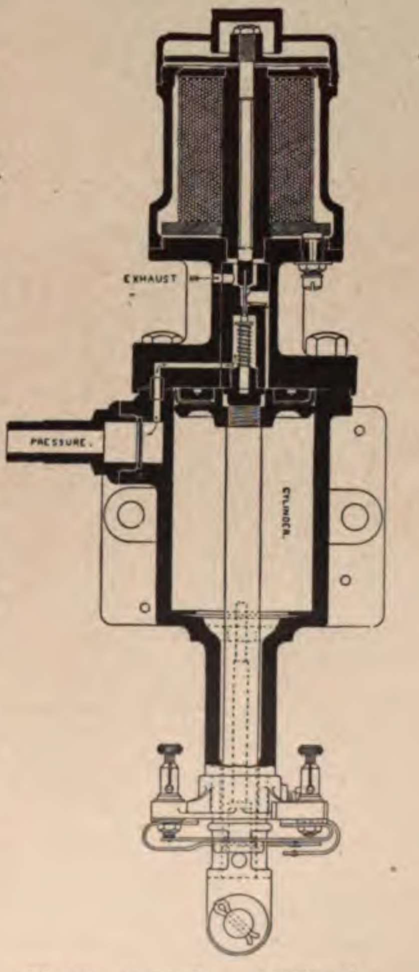


Fig. 154. Electro-Pneumatic Signal Mechanism.

148. Electro-Pneumatic Signals. The mechanical movement of signals by wires and rods is practically limited to about 2,000 feet and even at this distance it is troublesome. Electric power from batteries may be used when the power required is very small. An electro-pneumatic system uses compressed air whose power can be sent anywhere through pipes and which may be made to move not only signals but switches. The valves controlling the pistons are operated electrically by a current of low intensity, which may be provided by batteries but which in a plant of much magnitude is more economically obtained from storage batteries which are charged from a dynamo. The operation of the valve is shown in Fig. 154. In the position shown the magnet is *not* energized. When it is, the armature (at the top) is drawn down, which opens the conical valve just above the spring, and the air passes from the pressure pipe through the valve and down the passage alongside of the valve chamber until it bears on the top of the piston, which is shown in its extreme upward position. When the piston is forced down, it will raise the counterweight (see Fig. 155) and put the signal at "clear." When the magnet is de-energized for *any* reason, the spring forces the valve up, the air in the cylinder escapes through the exhaust and the counterweight not only raises the piston to the top but draws the signal to indicate danger. A failure of either the current or the pressure will thus put the signal at danger.

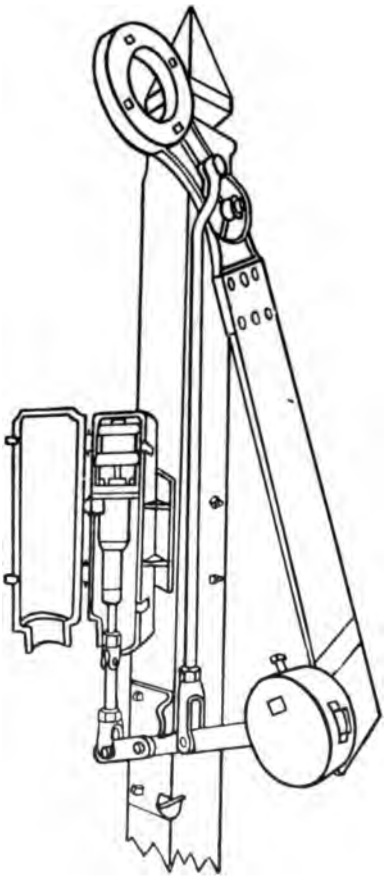


Fig. 155. Electro-Pneumatic Signal.

149. Electric Semaphores. Still another modification of automatic signals is the electric semaphore, which is a semaphore of the usual type, operated by an electric motor of about $\frac{1}{6}$ horsepower, the motor obtaining its current from a set of 10 to 16 Edison-Lalande battery cells which are placed in a box at the foot of the signal post. The motor winds up a light wire cable which raises the counterweight and thereby sets the signal at "clear." The motor is started and stopped by the action of a relay connected to the track circuit. The track circuit has the fundamental principles previously described, but has been made somewhat complicated so as to provide for the operation of distant as well as home signals and also the protection of and from all switches in the section. For the details of the track circuits the student is referred to the more complete works on this subject previously mentioned.

INTERLOCKING.

150. Principles. The interlocking of the switches and signals of a large terminal yard is such a complicated piece of mechanism that any adequate explanation and description would require too much space here. Nothing will be attempted but a demonstration of the fundamental principle. The reason for the necessity of interlocking is simple. A mere inspection of the design of a complicated yard will show that it is readily possible to arrange a large number of combinations of different switch movements for the operation of an equal number of trains simultaneously. But the operation of such switches is controlled from a signal cabin, and unless there are limitations on the combinations a signalman would be liable to set switches and signals so that two or more trains might collide. The fundamental principle of the interlocking device is comprised in the following statements:

(*a*) all switch signals are normally at danger;

(*b*) no switch lever may be set for any route until the switches for any other route which might cause a collision have been locked;

(*c*) the signal cannot be set to run through any switch until the switch itself is set.

Although an engineman may cause a collision by running past a danger signal, the worst that a careless signalman can do is to delay traffic. He cannot set signals and switches so as to cause a collision or even a "side swipe." The design of the interlocking machine must therefore be based on a study of the safe combinations, and then the interlocking machine must have its "cross locks" and "locking dogs" so arranged that no interference is possible. The case illustrated in Fig. 156 has purposely been made as simple as possible. The upper part shows merely the locking dogs (shaded full black) which are fastened on to the "locking bars" (which run crosswise) and the "cross locks" (shaded with cross hatching), which move at right angles to the locking bars. In the lower part of the figure are shown the signals and tracks for a crossover from a main track. No. 1 is the distant signal, No. 2 is the home signal governing the main track with respect to the crossover, No. 3 are the switch levers which work simultaneously, No. 4 is the signal governing movement from the siding to the main track, and No. 5 is the signal governing movement from the main track to the siding. No lever for a signal or a switch can be moved without simultaneously moving the locking bar (having the corresponding number) from right to left as shown in the figure.

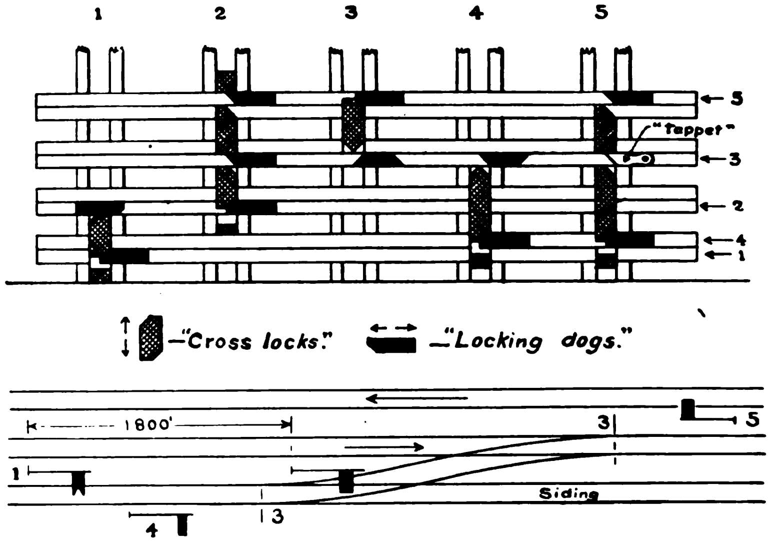
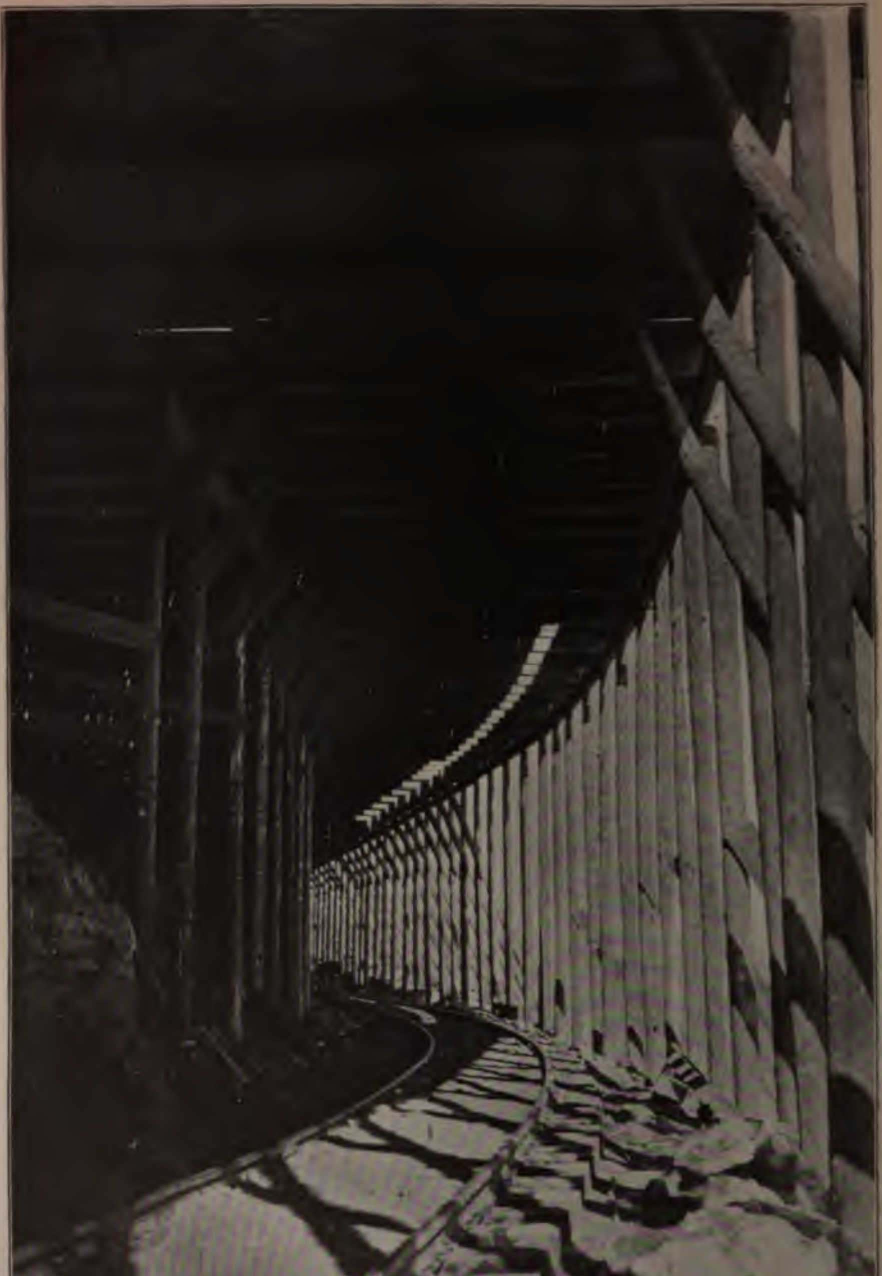


Fig. 156. Interlocking.

The wedge-shaped ends of the locking dogs will move (if possible) the cross locks with which they may come in contact. If any cross lock is immovable because it is already in contact with some other locking dog, then it will be impossible to move that lever until the lever (or levers) controlling all interfering locking dogs have been so moved as to remove the obstruction. The position of the locking dogs in Fig. 156 is that for all signals normal or at "danger." Suppose it were attempted to "clear" signal No. 1. To do so, locking bar No. 1 must move the cross lock No. 1. But this is impossible since one of the dogs on locking bar No. 2 interferes. Lever No. 1 cannot therefore be moved until



INTERIOR OF A RAILWAY SNOW-SHED IN THE ROCKY MOUNTAINS

Wooden construction. Many miles of such structure are necessary to keep the roadway open in spite of snow-slides and drifts.

lever No. 2 has been cleared, which operation will move that dog far enough to the left so that the cross lock can move up. And this is in accordance with the principle previously stated that a distant signal should not be cleared until its home signal is cleared.

As another illustration, when the signal No. 2 has been cleared, the locking bar No. 2, by means of its attached dogs, moves the cross lock No. 2 upward. This cross lock is then set against locking dogs on each of locking bars No. 3 and No. 5 and prevents them from being cleared. Of course the signals for the crossover should not be cleared while the signal is set for a clear main track.

Exercise. The student should draw a modification of the upper part of Fig. 156, as it would be placed to indicate that the switch was set for a crossing from the siding to the main track. It should be noted that signal No. 4 is set "clear" when it is designed to move along the siding without using the switch, and No. 5 is set clear when the switch is set so that a train could run backward past the switch without using it. Both No. 4 and No. 5 are set at "danger" when it is designed to run from one track to the other.

The cross locks No. 1 to No. 4 inclusive are each in one piece with notches cut for the dogs. Cross lock No. 5 has the upper part separate. When the lower part is moved it does not move the upper part unless the "tappet" on locking bar No. 3 has previously been moved between the parts. The tappet is unnecessary with the simple combination of levers shown, but might be necessary with a somewhat more complicated system.

Of course the above description makes no mention of a multitude of details necessary for a manual machine, to say nothing of the complication required for an electro-pneumatic interlocking machine. But whatever the complication or how many may be the number of levers, the interlocking principle is as above.

TRACK MAINTENANCE.

151. Tools. Tools should be of good quality and well designed for their use. Economy in this respect, to save initial cost, is apt to increase the labor item and since the labor costs over 60 per cent of the total cost of track maintenance, a very little discouragement of labor owing to inefficient tools would more than overbalance any possible saving in cost. The list of tools required for the varied work of a track gang is quite large, and therefore an effort should be made to pare down the list as much as is practicable or safe, because it is correspondingly difficult for a track foreman to prevent losses due to carelessness. The following list is based on the requirements of a gang of six trackmen and a foreman.

A large proportion of the tools are for work on which not more than one or two men need work at any one time. When the list calls for six or more of any one tool, they are always the tools which are in constant or excessive use, or which are liable to become quickly broken, and of which an extra supply is a necessity for use while waiting for requisitions to make up for loss or breakage. The list is taken, with some slight modifications, from Camp's Notes on Track.

Adzes	2	Grindstone	1	Scythes—grass	4
Ax—chopping	1	Hammers—spike	4	" —brush	4
Ax—hand	1	" —sledge—16 lbs.	1	Snaths	4
Auger, 2-inch	1	" —strike—10 lbs.	1	Shovels—track	8
Bars—claw	2	" —nail—claw	1	" —scoop	4
" —crow	0	" --ballast*	6	" --long handle	1
" —pinch	6	Hatchet	1	Saw—hack, blades	12
" —raising	1	Hoe--garden	1	" " frame	1
" —tamping	8	Jack—track	1	" hand	1
Brace and bits	1	Key—switch	1	" crosscut	1
Brooms (coarse)	2	Lanterns—white	2	Screwdriver	1
Brush hooks	2	" —red	2	Spade	1
Car—hand	1	" —green	2	Steel square	1
" —push	1	Level board	1	Tape (50′, graduated to tenths)	1
Car chains	2	" --spirit--pocket	1		
Chisels—cold	2	Locks--switch--extra	2	Tongs--rail	4
" —track	12	Mattocks	2	Tool box	1
" —wood	1	Oil can 1 gal	1	Tool checks	6
Curving hooks	2	" " —2 "	1	Torpedoes (with box)	24
Chalk line	100 ft.	Oiler--squirt	1	Verona spike puller	1
Ditch line	150 ft.	Padlocks	2	Vise	1
Drawshave	1	Picks	8	Water pail or jug	1
Dippers (or cups)	2	" --tamping*	8	Weed scuffles	6
Files	3	Punch--hand	1	Wheelbarrows	3
Flags--red	4	Rake--garden	1	Whetstones	4
" —green	2	Rail drill	1	Wire stretcher	1
Forks--ballast*	4	" " bits	6	Wrenches—track	3
Gauge	1	Rule--two-foot	1	" —monkey (8″)	1

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* Needed only in stone ballast.

The first comment on the above list is in regard to the bars of various kinds. *Claw bars* are used for spike pulling. The ideal design is one that would permit pulling the spike with one stroke without changing the fulcrum, and that will also pull it clear out without bending. Apparently this is mechanically impossible and in spite of the many efforts which have been made and the new designs which have been brought out, the old "bull's foot" claw bar seems to be the best.

The "*Verona spike puller*" is attached to a spike which is in a confined place (such as behind a guard rail) and is operated by means of an ordinary claw bar resting on top of the rail.

"*Crow*" *bars* are considered to be those which taper down symmetrically to a wedge-shaped edge at the so-called "point," in contradistinction to "*pinch*" *bars* on which the chisel edge is even with (or outside of) the face line of the bar. The number of crow bars is put at "0" to emphasize Mr. Camp's opinion that the crow bar form should not be used and that the pinch bar form is far preferable.

Tamping bars should not weigh more than 10 pounds nor should they be more than 5 ft. 3 in. long. If the handle is solid it is rather small and hard to hold. It is therefore sometimes made as a pipe, with a malleable tamper. Another form uses a wooden handle.

Track chisels are cold chisels provided with a handle of wood by which they may be more readily and safely held in position. They should be about 1½ in. square and 8 in. long, made of tool steel. A single blow may break them or render them useless until re-tempered and re-ground, and therefore a large number is necessary.

Gauges. These may be divided into three classes. The first is the "home-made" type of wooden gauge which is perhaps brass bound. One common objection to this form consists in the danger that it will not always be placed truly at right angles to the track. The effect of this error is to make tight gauge. To obviate this error, the "Huntington" track gauge has at one end two lugs about seven inches apart and one lug at the other end. The gauge is the distance from the single lug to the middle point of the seven-inch line, the two lines being at right angles. The device is theoretically perfect provided that the two lugs at the one end are both in contact with the head of the rail. A slight error in this respect will make the gauge too wide. The "Warren" gauge has two short circular arcs forming part of a complete circle, whose diameter is the gauge, fastened to the gauge bar.

Hammers. For section work, spike hammers should not weigh more than 8 pounds and have a length of about 10¾ inches. The 16-pound sledge is only needed for occasional very heavy work, when it is however almost essential. The 10-pound striking hammer is the better one to use with track chisels rather than to use the spiking hammers as is so frequently done. The ballast hammers are only used for breaking up stone for ballast and are unnecessary even for this purpose if machine broken ballast of uniform size is furnished.

Track jack. One of these is illustrated in Fig. 107. They are certainly handy and economical tools for the track gang, but more than one serious wreck has been caused by the inability of the gang to remove the jack before the arrival of an unexpected train, and as a derailing device a jack is exceptionally effective. Track instructions generally specify that they must not be placed between the rails.

Level board. Such a board usually has a level tube sunk in the upper edge. At one end a series of steps are cut, each with a base of about two inches, and with risers of one-half inch, beginning at the lower edge. The discussion on the superelevation of the outer rail (see § 119) shows the foolishness of over-refinement in such work. If the required superelevation is 2.5 in., the fifth step of the board may be placed on the outer rail and the plain end on the inner rail. When the track is properly adjusted the bubble should be in the center. Of course the adjustment of the level bubbles should be carefully watched and frequently adjusted if necessary.

Shovels. The best shovel for track work is the short handled shovel with square point, made out of a single piece of crucible steel. The blade should have a length of about 12 in. When this has been worn down to 9 in. it should be thrown away—for track work. Its use is then uneconomical. The scoop shovels are for handling cinders and packed snow. The long-handled shovel is for digging post holes. It should be round-pointed.

The above list includes only the tools which will be required by almost any track gang. Cant-hooks and peavies are frequently necessary for handling timber. Blasting drills, wedges, powder and fuse are sometimes needed to break up masses of rock which may have fallen into a cut. Culverts and bridge channels get choked up with timber and debris of various kinds which may need ropes and tackle to clear them. A jim-crow rail bender is occasionally necessary, although one such may be made to serve two or more section gangs.

152. Work Trains. The work of a track gang is usually confined to one "section," which is usually not more than five miles long, and which on roads of the very heaviest traffic may be shortened up to a mile. On exceptionally poor light traffic roads, they are made eight and even ten miles. For ordinary work their hand car and push car furnish all needed transportation facilities for themselves and materials. But there is much work which is more irregular in its character, which must be handled on a larger scale, and which requires for economy a work train. Such work is the distribution of track materials such as ties, rails and ballast from the sources of supply to the places on the road where they are needed. Also, when re-ballasting is to be done on an extensive scale, when heavier rails are to be substituted throughout, or, in short, when there is any work to be done which is beyond the routine work of keeping the track up to its normal condition, then a work train with its usual force of laborers can accomplish the work with greater economy.

The work train is usually hauled by the worst engine on the road, sometimes by one which would otherwise be sent to the scrap heap. Whatever the justification of this policy, it may be carried so far that the regular train service suffers by the inability of the work train to keep out of the way of regular traffic, or else there is the false economy of wasting the time of the work train gang while trying to save by utilizing a worthless engine. A passenger engine which may have proved too light for regular service is preferable to a freight engine, since the work train should be capable of making good speed in running to a siding and the load is usually light.

The minimum requirements for the train should include a large caboose and a flat car provided with large tool boxes for picks, shovels, bars, hammers and other track tools. Underneath the caboose may be hung a large box in which may be stored ropes, pulley blocks, chains, jacks, etc. Since the cost of train crew wages, fuel, and other expenses which must be charged up for the use of the rolling stock will aggregate about $25 per day, there should be enough laborers attached to the train, and their work should be so planned as to justify this additional expenditure.

The minimum number of laborers should be about 20, and this should be increased to as many as can be profitably employed. Since the work of the train is scattered over a great distance, the company must choose between wasting considerable time both morning and evening while carrying the gang to and from their homes, together with many miles of train service, or of providing boarding cars, provided with bunks and one or two cars for kitchen and dining cars. One large, clean box car can be easily and cheaply fitted up as kitchen and dining car for 24 men. If the crew is much larger, one car should be devoted to kitchen and the storage of supplies and another car used for a dining car. An ordinary box car, or an old passenger car can be readily fitted up with four double lower berths and four double upper berths on one side and four lower and four upper single berths on the other side, thus accommodating 24 men. Even better accommodations may be provided when the need for such a train and gang is so regular that it will have practically permanent employment. A little extra money spent by the company in providing comforts for the men is immediately repaid in a better quality of work and less straggling off.

153. Ditching. While the routine clearing up of ditches is part of the work of a section gang, it will frequently happen, especially when the slopes have a disintegrating soil, and also when the slopes have been made originally too steep, that the winter's frosts will fill up the ditches to such an extent that it is best taken out with a work train gang. Ordinarily the section gang would need to load such material on their push car or on wheelbarrows and run the material out to the end of the cut where it may be harmlessly wasted. If the cut is very long, such hauling would be very expensive. Since the regular schedule will not usually permit the train to stand long on the main track, especially on a single-track road, the loading must be done in the shortest possible time. This usually implies that a part of the gang should remain at the cut while the train is running off to unload and that they should all work there if the train must run to a siding merely to let a regular train pass. During such times the men can scrape down all loose material from the side slopes and loosen up the filling in the ditch, so that it is all ready for shovelling when the train arrives.

When the cuts are not very deep, such material is sometimes thrown up on the top of the bank, even by using a temporary staging on which the earth is thrown and then again shoveled to the top of the bank. In any such case the earth should be thrown well back from the edge of the bank so as to guard against its being again washed into the cut. It also should not interfere with the surface ditch which should have been cut on the top of the bank to prevent surface water from the slope above from running down into the cut.

154. Distributing Ties. The methods to be used necessarily vary with the sources of supply. If ties were obtainable from farmers and were delivered along the right-of-way on every section of the road, very little if any distribution by a work train would be necessary. When, as the other extreme, there is no local source of supply, the ties must be hauled many miles and so distributed that subsequent distribution by the trackmen will be reduced to a minimum. Since economy requires that ties shall only be replaced by an actual count of those which are defective, an essential preliminary is that a marker shall be placed along the the track for every ten ties required, or that the number required between two consecutive telegraph poles shall be marked on the poles so that it may be seen as the train approaches. By this means ties may be thrown off as required while the train is moving at a speed of about six miles per hour. On light traffic roads the work of tie distribution is frequently done by the local freight train. While this may be and often is the best policy, the cost per tie is much greater.

155. Distributing Rails. The method of handling rails depends very largely on the cars on which they are loaded, and also on their length. They are dropped off most easily when loaded on to flat cars, but frequently they are loaded on to gondolas and even in box cars by making a hole in the end of the car. Rails of 45 and 60 feet can only be loaded on to two consecutive flat cars. If they are being unloaded in one place simply for storage, a derrick of some kind, even though temporary, is wise economy. For distribution along the track they are either dropped over the side of the car or pulled off from the end. If they are dropped over the side they are apt to be kinked. Sometimes they are slid off on skids made of two timbers or pieces of rail about 10 feet long, but this is impracticable in some localities and it lands the rail at some distance from the track. The car to be immediately unloaded may be placed at the extreme rear of the train. Then a rail hook attached to a sufficient length of rope may be hooked into one of the bolt holes and the rail may be drawn off the end. By placing a "dolly" on the end of the car the rail may readily be drawn off by hand. As soon as the center of gravity passes the dolly, the outer end falls easily to the track and then, pulling the train ahead, the other end is let down easily as it drops off. Sixty-foot rails are so flexible that a considerable part of the length will be resting on the ground before the other end leaves the car, and will not be injured by dropping on the ties.

When the rails are especially long and heavy, an easier method is to hook on the rail-hook and attach the ropes to a track rail or around a tie. Then let the train move ahead until the rail is drawn off. Even the dolly under the rail at the end of the car is unnecessary with this method. If rails are needed for both sides, two such ropes and hooks may be used simultaneously. With a little more care this may be so done that the end of each rail comes almost exactly at the required joint, even allowing for staggering the joints on the two lines of rails. By attaching a push car to the car carrying the rails, the rails may pass over that and down on to the ground without any danger of injury from the drop.

The reverse operation—loading old rails onto a car—is heavy and costly work when done by hand. The best plan is to do it by means of a derrick. If it must be done without such aid, it facilitates the work to make an inclined plane by attaching a push car to the flat car on which the rails are to be loaded, and then by placing several dollies on this plane, the rails may run up on rollers with a minimum of actual lifting. It might be thought that a 70-lb, rail, 30 feet long, which weighs 700 pounds, should not be an excessive load for six men. When the men are carefully drilled to lift together and simultaneously raise the rail above their heads and throw it with machine-like precision on to the car, it may be (and is) successfully done in this way, but if one or two men shirk or do not lift with the others, the load is concentrated on the others. They successively become frightened and, to save themselves, "jump from under"; the remainder cannot sustain the load and it falls. It is lucky if someone does not have a foot crushed. The longer and heavier rails cannot be handled in this way.

156. Handling Ballast. A railroad must consider itself unfortunate if it does not have a gravel bank at some place along its line. The bank generally extends into the adjoining property, which is either bought outright or the gravel privilege is bought. The last method generally specifies that the top soil shall be reserved and spread upon the excavation after the gravel is exhausted. The gravel is usually overlaid with more or less vegetable soil. Sometimes the amount of this is so insignificant that its presence may be ignored, but if the depth is appreciable it will pay to strip it. A spur track whose minimum length is the length of the train must be run off from the main track. The method of attacking the bank depends on the method of digging—whether by steam shovel or by hand digging and shoveling.

About twenty cubic yards per day may be considered a fair day's work in loading gravel cars at the pit. A steam shovel with a dipper holding 1½ to 2 cubic yards can load 800 to 1,200 cubic yards per day, depending on the prompt handling of the cars when loaded. Even this figure has been greatly increased under exceptionally favorable conditions. But the use of a steam shovel implies the use of a locomotive, which must be constantly at the pit shifting the cars so that there is a car constantly in place within range of the shovel. The cost of running such a shovel with its attendant locomotive will be about $50 per day. This will pay about 40 laborers who could dig about 800 cubic yards. Therefore, unless the circumstances are so favorable that the shovel can exceed 800 cubic yards per day, the work may be done about as cheaply by hand shoveling. This is, however, about the limiting case. With good management a large shovel can take out gravel much cheaper than it can be done by hand. 20 cubic yards per day at a labor cost of $1.25 per day makes the gravel cost about six cents per cubic yard loaded on the car at the pit. The average cost of such hauling on a Western railroad was computed by the management to be 0.35 cent per cubic yard per mile. In this case the quantity handled was very large and the cost may be considered exceptionally low.

When the work is done on a small scale, and especially when the gravel is loaded by hand, hand methods would be used for unloading, but there is great economy in the use of a plow for unloading. This implies the use of flat cars, which are in fact almost universally used for ballast work—barring the special patented ballast cars. The plows are "center unloading" or "side unloading," and some of the most recent forms are adjustable so that they will unload all to either side or will unload to both sides in any desired proportion. The plow is drawn over the tops of the cars by a cable. The cheapest method is to stop the train where desired, set the brakes, uncouple the locomotive and attach to it a 1¼″ or 1½″ wire cable. Commencing with the plow at the rear car the locomotive moves ahead and draws the plow over all the cars. This method has many objections, especially when it is done on curves. A much better method is to have a car carrying a hoisting engine, which may be supplied by steam from the locomotive by a flexible tube, if the car carrying it is placed immediately behind the locomotive, or preferably which is supplied from its own boiler placed on the car. A wire rope from this engine hauls the plow. One great advantage of this method lies in the fact that the train can be kept moving if desired while the plow is working.

If it is desired to distribute less ballast per car length than the car load, it may be done by moving the train ahead at just such a speed that will give the desired result. Incidentally, this method is very useful when making a fill from a trestle or to fill up a washout. By putting the plow at the rear of the train and drawing the plow backward, the speed of the train and of the plow can be so regulated that the material will be deposited with as great concentration as desired. If it is desired to fill up a hole, the whole train load may be deposited in one spot by simply hauling the plow back as fast as the train moves forward. The average cost of thus unloading with a cable has been computed as about one-half cent per cubic yard. During recent years many styles of ballast cars which are easily and automatically unloaded have been placed on the market. Some of these have been designed with the distinct idea of being used in connection with local freight trains. They are picked up at the gravel pit by a local freight going in the desired direction, are hauled to places along the road which have been previously marked with stakes, are dumped with a delay of only a minute or so, then hauled on to where they may be sidetracked and hauled back to the pit by another local freight. Gondola and coal cars having hopper bottoms are also used extensively for hauling ballast.

157. Trestle Filling. This has become a very common form of work for the work train. When the construction of a railroad is once definitely decided and work is begun, any measure which will hasten the opening of the road for traffic has a very high money value. Therefore trestles have been built where embankments are a better form of permanent construction. The preliminary construction of trestles is further justified by the fact that the immediate construction of an embankment would often involve very expensive hauling with teams from borrow pits in the neighborhood, while a future fill may be made by the train load, as described below, at a much less cost. Incidentally, time is allowed to determine the maximum water flow through the hollow crossed by the line, and the size of the culvert required may be more accurately determined. The cost of the culvert, which may be very considerable, is also deferred to a time when the road can better afford it. At the time that many existing trestles were built the cost of timber in their localities was so small that the trestle may have been actually cheaper.

Many roads are now confronted by the necessity of either replacing the trestle or filling in with earth. While the relative cost is very variable, depending on the local price of timber, the proximity of a sufficient supply of available filling and the methods to be employed, yet as an approximate figure it may be said that fills as high as 25 feet may be filled with earth as cheaply as a trestle can be reconstructed. But when it is considered in addition that the average amount of timber required annually for repairs of trestles is about one-eighth of the volume, also that the labor involved in maintenance is very great while it is almost insignificant on an embankment, also that the danger of accident on a trestle and the disastrous results of a derailment which may occur on a trestle is so much greater than on an embankment, the height at which it becomes economical to fill with earth instead of reconstructing the trestle increases until it may reach 50 feet. But the filling in of high trestles involves several special constructive features. The hollow may have at the bottom a very soft soil which cannot sustain a heavy embankment without considerable settlement. Such a settlement will prove destructive to almost any culvert unless a solid foundation may be made for it. Under such conditions a pile or concrete foundation for the culvert may become a necessity.

The dumping of earth and particularly of boulders, stumps and clods of frozen earth may do serious injury to the trestle unless means are taken to guard against it. This may be done by placing an "apron" on each side which will deflect the earth so that it falls outside the trestle. As the piles grow on each side the intermediate space will be filled up. The longitudinal braces which are most apt to suffer are sometimes strengthened by heavy timbers, which may be old stringers, etc. The filling should be done regularly along the length so that the bents will not be forced out of place by an unsupported pressure of earth on one side. If the bank is formed merely by dropping earth loosely from above, its slopes will be steeper than can be retained permanently. The result is frequently a disastrous slip. This feature justifies the spreading of the earth by scrapers as the filling proceeds. This method has the additional merit of packing the earth so that there is almost no settlement and the stringers may be pulled and the ballasted roadbed may be constructed very soon after the filling is complete. Otherwise the settlement is so great that six months or a year must elapse before track laying is permissible. During this time the embankment may settle 10 per cent. This earth-spreading may be done for two or three cents per cubic yard.

The choice of filling material is an important matter. A sandy or gravelly soil is the best. Clay is apt to be very troublesome, for, no matter how hard it may be in dry weather, it will slip and run when it becomes wet. This is especially true when the base of a fill is on a steep side slope. In this case the whole fill may slide down the hill. One means of preventing this is to dig trenches along the slope. Even plowing the surface in contour furrows may be sufficient to prevent such a slip. The material for such a fill will usually come as the spoil from a widened cut, loaded perhaps with a steam shovel into dump cars or on to flats from which it is scraped by a plow, as previously described.

The practice of immediately planting tufts of Bermuda grass and even tree slips which will take root and grow and thus bind the embankment together as well as cover it with a surface of sod which will protect it from rain-wash is a measure of true economy which always pays. The total cost of such a fill must combine the cost of loading, hauling, spreading (if it is done) and the other expenses incidental to making a finished embankment, but the record made by many roads on these items show that it may be done at very much less cost than by the methods which are usual or possible during the original construction of the road.

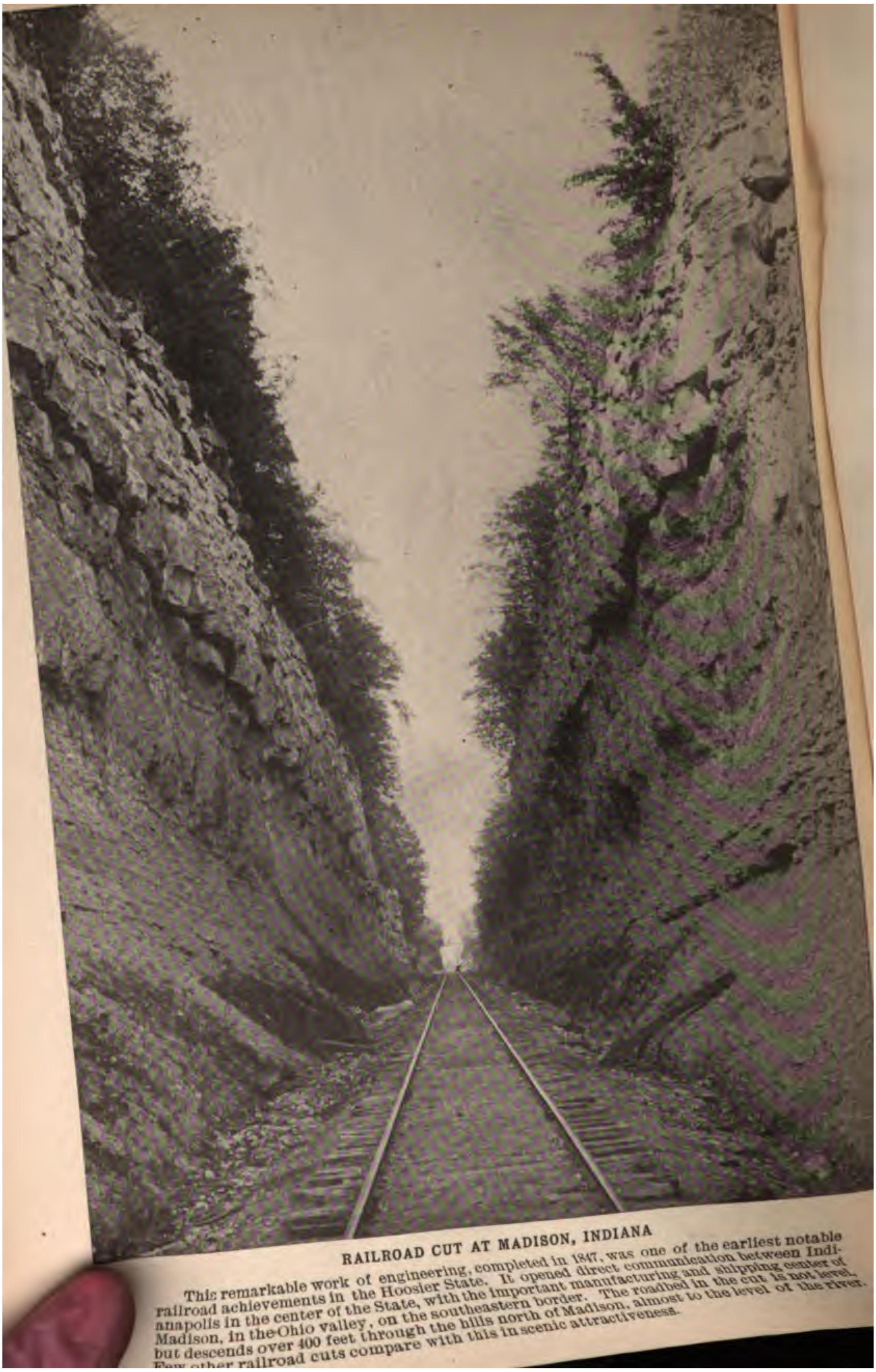
158. Organization of Track Maintenance Labor. Although there is much variation in the practice of roads as to the succession of authority among the higher officials of the road, there is a very general agreement in placing the immediate supervision of the track for each division of approximately one hundred miles under a man known as roadmaster or perhaps supervisor. Some roads extend the authority of the roadmaster over a greater length of road and then appoint "supervisors" who individually control shorter lengths and who report to the roadmaster. The supervisor of each minor division superintends the work of the several section gangs in his division.

The roadmasters usually report to a division engineer, but except in matters of exceptional importance or which may involve new methods of work, the roadmaster is expected to do his routine work without special orders and to be responsible for its proper execution. The roadmaster should be thoroughly conversant with every phase of the work done under him, and although it is preferable that he should have come up from the ranks, he should have a far better education than is possessed by the large majority of track laborers. The best roadmasters are those who have a technical education but who have served a sufficient time in the ranks to have become familiar with the practical details of track work. The Southern Pacific R. R. Co. require a roadmaster to "pass over the entire straight portion of his districts, either on foot or on velocipede cars, at least twice every month, and over that portion in canyons and in the mountains at least three times per month." He should have the work of the entire division so thoroughly mapped out in his mind that he has a sufficiently clear idea of the condition of every part of his division at any time and thereby save himself from censure due to any neglect of the track work at any place.

The most effective way to do this is to have the section foremen under such a state of drill that there will be no failure to remedy any slight defect or report a greater one. The roadmaster must rely on his discipline of the section foremen rather than on his personal observation, although he should not relax any effort to make his personal observations as thorough as possible.

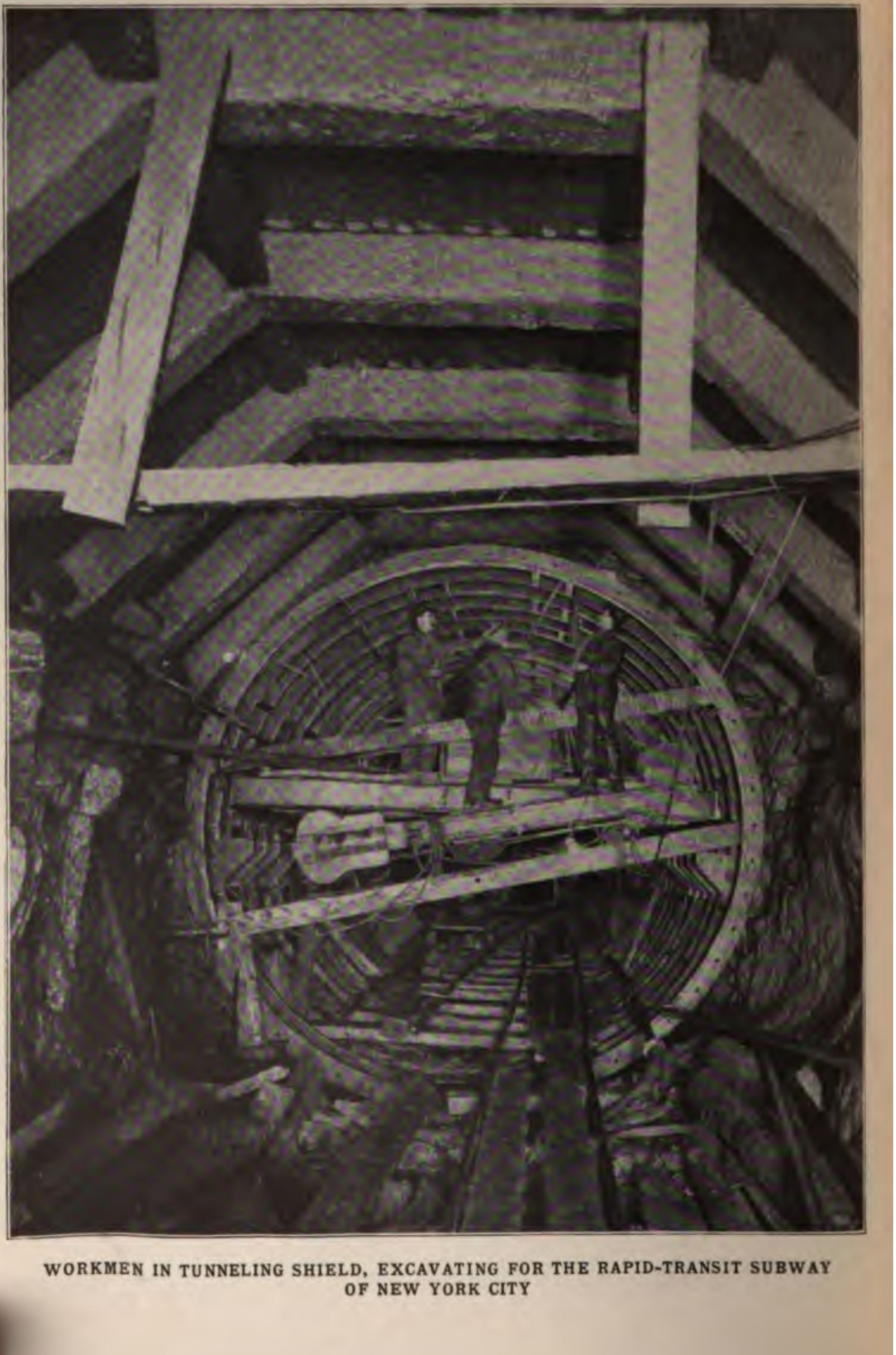
The section foreman should generally be a man who has served his time as a track laborer, but he should also be a man who has sufficient education and intelligence to make out reports and correctly interpret plans and tabular statements. Another absolutely essential quality is an ability to control men without violence or abuse. He should not only thoroughly understand all the details of maintaining a track in condition, but should be able to repair a track and make it safe for trains in any ordinary emergency such as a broken rail, a washout, or a tearing up of the track due to a wreck. He should familiarize himself with all rules of the road regarding train running with which he may ever be immediately concerned, and also with all rules and standards of track construction which may have been adopted. The last qualification, which is becoming more and more essential, raises the standard for section foremen above what was formerly considered necessary.

Many roads require (by their rules) that the foreman shall take part in the manual labor of the gang. The wisdom of this rule depends somewhat on the work being done and on the number of men in the gang. If there are as many as eight laborers in the gang, the foreman may have all he can do in directing them. It is frequently advantageous to have the men work in pairs, and when the work is light, it may be best to have five laborers with the foreman in a gang, and then the foreman may work with the odd man.



RAILROAD CUT AT MADISON, INDIANA

This remarkable work of engineering, completed in 1847, was one of the earliest notable railroad achievements in the Hoosier State. It opened direct communication between Indianapolis in the center of the State, with the important manufacturing and shipping center of Madison, in the Ohio valley, on the southeastern border. The roadbed in the cut is not level, but descends over 400 feet through the hills north of Madison, almost to the level of the river. Few other railroad cuts compare with this in scenic attractiveness.



WORKMEN IN TUNNELING SHIELD, EXCAVATING FOR THE RAPID-TRANSIT SUBWAY OF NEW YORK CITY

RAILROAD ENGINEERING.

PART III.

ECONOMICS.

Railroad Finances.

159. Capitalization. Practically all of the following statements regarding capitalization, etc., of the railroads of the country are taken from the reports of the Interstate Commerce Commission and may therefore be considered as reliable as any which are obtainable. Many of the following figures are taken from the report for the year ending June 30, 1902. At that time the capital stock was given as $6,126,635,752. Disregarding the fractions of million, the funded debt, expressed in millions, was 6,446. Other liabilities, with "profit and loss," make up a total of 14,270 millions. This represents approximately one-sixth of the national wealth. Considering this value as "assets," 10,021 millions of it is set down as "cost of road" and 637 millions as cost of equipment. Unfortunately the finances of roads have been so manipulated that the statement of "cost of road" does not represent the capital actually spent on construction.

The above figures apply to 187,442 miles of line, the total mileage being over 201,000 miles. The railroads directly employed 1,189,315 employees, to whom they paid over $676,000,000, which is over 39% of the gross receipts, which were over $1,726,000,000. The employees represent a population of perhaps four or five millions who are directly dependent on the railroads for support. Considering the industries such as locomotive and car shops which depend entirely on railroads for business and also the industries such as steel mills and bridge works which have railroads as their largest customers, it may perhaps be estimated that one-fourth of the entire population are directly or indirectly dependent for their support on railroads.

If the argument is carried still further and the fact is recognized that a very large proportion of the products of agriculture,

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mines and manufactories could not be transported to consumers, would not be utilized and therefore would not be produced, the debt of the country to railroad transportation may be better appreciated.

Although the stocks and bonds of many of the smaller railroads are owned by large corporations in their corporate capacity, yet of the total of 11,237 millions of stocks and bonds outstanding in 1902, 9,029 millions, or over 80 per cent, were owned by "other than railroad corporations"—chiefly private investors. On the basis of total assets of 14,270 million dollars and an estimated population of 78,595,000 people, the *average* ownership is over $181 per head of population. The total revenue of over $1,922,000,000 represents an average payment of over $24 per inhabitant for the year. The "number of passengers carried one mile" was 19,690,000,000, which means that the *average* passenger travelled 250 miles during the year. The average number of passengers on a train was 45 and they travelled an average journey of 30 30 miles.

The "number of tons of freight carried one mile" was 157,289,000,000, which means that the *average* inhabitant supplied a freight business equivalent to moving 2,000 tons one mile, or moving one ton 2,000 miles, or moving 50 tons 40 miles. As an aid to grasping this perhaps incredible statement (combined with the statement of an average annual payment of $24 per inhabitant) it should be remembered that whenever a ton of coal or even a pound of sugar is bought, the price paid includes payment made to a railroad company for freight.

While the above figures must be considered simply as averages, and not necessarily applicable to any one road, they give some idea of the magnitude of railroad business and what the *average* railroad may be expected to do. Considering, however, that the great railroads of the country are already built and that the roads yet to be built will probably be of minor importance, even such an average statement could hardly apply to any new enterprise except with a very large discount.

160. Stocks and Bonds. An ordinary mercantile business or even an ordinary factory conducts its business on the capital directly furnished by the owner or owners. Therefore any profit over the operating expenses may be applied as dividends no matter how small the percentage. Very few of the railroads of this country have been constructed, even approximately, on this basis. Usually a large part of the virtual ownership of a railroad is represented by bonds. The limit of the issue of the bonds may be that which expresses the confidence of the public in the enterprise, or, in other words, the value which it is assumed that the whole property could be sold for under a foreclosure sale.

During the early history of railroading, when railroads were being run through well established communities which were without railroad facilities, the success of the enterprises seemed so certain that little or no difficulty was experienced in borrowing on bonds enough (and even more than enough) to construct and equip the road complete. But such opportunities are practically past. The capital stock actually paid in represents the margin or the uncertainty between what it actually will cost to build the road and its estimated foreclosure value. The nominal issue of stock is usually about equal to the value of the bonds.

At its best, the inception of such an enterprise means a considerable outlay of money. A group of men, acting on the belief that a road passing through certain towns will be a profitable enterprise, form a temporary organization, develop the enterprise, have surveys made, and then if the developed plans still look encouraging, have bonds engraved and placed on the money market for sale. Even if it were possible to raise enough money for actual construction, the amount of money required for this preliminary work, although but a small precentage of the gross amount required, is sometimes a large sum of money.

The gross amount required is increased by the frequently ignored fact that a road does not attain its "normal" traffic for five or ten years after it begins operation, and unless it has sufficient funds as "working capital" to tide over the initial period when it does not perhaps pay operating expenses, it is very apt to go into the hands of a receiver.

Stocks and bonds may therefore be considered as representing two forms of ownership. The interest on the bonds is a lien on the receipts, after the operating expenses have been paid. Such interest must be paid in full before any dividends on stock may be paid. The security of the bonds is therefore comparatively good and the profit comparatively certain although it is small.

On the other hand, the stocks are much more speculative. No dividends are paid until the operating expenses and the bond interest is fully paid, and if this is not paid, the bondholders have a right to demand that a "receiver" be appointed and if necessary that the road be sold. Since such a sale will not usually realize more than the face value of the bonds (and sometimes not even that) the stockholders may lose their entire investment. But if the road makes money, the excess which may be allowed for dividends may be a very large return on the amount of capital *actually paid in.* It may very easily be shown that a comparatively small change in the amount of business done may suffice to change a good profit for the stock holders into an actual deficit which makes a receivership dangerously probable.

The relative profit on stocks and bonds and the fact that railroad securities, although sometimes very profitable, are very precarious in value is shown by the following statements: The year ending June 30, 1902, was the best year (up to that time) ever known in the railroad business. But, in spite of this, 44.6 per cent of all railroad stocks then in existence paid *no* dividends. The average rate paid on dividend-paying stock was only 5.55 per cent. Even granting that much of railroad stock is "watered"—which means essentially that it represents little or no cash actually paid in—the fact remains that during that year 44.6 per cent of all the stock issued paid no dividends. From 1895 to 1897 over 70 per cent of all railroad stock paid no dividends.

The record regarding bonds is much better, the percentage of the entire bond issue which failed to pay anything during 1901–2 being less than 5 per cent. While it is true (almost without exception) that a railroad builds up the section of country through which it passes and increases its value far beyond the cost of the road, yet it is also true that very few roads which are old enough to have a history have escaped a receivership at some time in their history, even though they may now be gilt-edged properties.

161. Gross Revenue. The estimation of the probable volume of traffic or the gross revenue of a proposed road can only be approximated at best and even this requires experience. Since it requires five years or more for a road to attain its normal traffic, investcrs should not be disappointed when the returns for the first few years are less than those anticipated. There are three general methods:

(*a*) The annual reports of the Interstate Commerce Commission give the gross earnings from operation for the road in each of ten sections into which the country has been divided. Dividing this gross value by the population of the section (which is deducible from the report) an *average* value per head of population for that section is obtainable. The value for the whole United States is, as previously stated, over $24, but the value for some one section may prove quite different from this. Multiplying the value obtained by the population which may be considered as tributary to the route of the road, we have a very approximate value for the income of the road. The two obvious weaknesses of the method are that the receipts of the proposed road may prove very different from the average for that section and also that the computation of the tributary population is a very uncertain calculation. But since the method may be easily tried, it furnishes a check of some value.

(*b*) Usually there are one or more roads which may be selected which have substantially the same characteristics and whose incomes per mile of road are nearly equal and which supposedly equal the expected income of the proposed line. Assuming the existence of such roads and that the engineer has sound judgment in estimating their characteristics, this method is one of the best and should be employed if possible, at least to check the value of any other computation.

(*c*) The best method (when well done) is to study the resources of the belt of country which will be tributary to the proposed line, estimating the business obtainable from every factory, mine, blast furnace, farm, village, etc. When, as is usual, the line passes through or reaches cities which are already supplied with railroad facilities, the detailed computation of business becomes very uncertain. But if the chief business of the road is to develop local business along a route which has no other means of communication, then the computation is easier. The two dangers in the method lie in the entire neglect to allow for certain important sources of income and, on the other hand, to overestimate the income from a certain source. Analogous to the last is the neglect to allow for present or future competition, which may practically cut off sources of income.

Although some idea of the product of factories and mines may be obtained from records as to their present or prospective output, the income from passenger business can only be computed from comparisons with other roads. The freight business is generally two-thirds of the business of a road, except on those roads which have an enormous suburban traffic. Although the average receipts per passenger mile is about two cents, it is the enormous commuter business and the growth of travel on 1,000-mile tickets which brings down the average to this figure from the usual charge of three cents per mile and the even higher charges on roads with light traffic and very heavy expenses.

The number of passengers per train is of course very uncertain. The average number of passengers carried for each passenger train mile, as previously stated, was 45, which is less than a car load. And when it is considered that even this average includes the heavy traffic roads and the well filled trains on suburban roads, the average number on a light traffic road must be very small. The number of passenger trains per day bears but little relation to the number that can be carried in one train load—as the above (45) shows.

The passenger business must be developed, coaxed and encouraged, which can only be done by a frequency of service which is usually far ahead of the requirements from a mere hauling standpoint. It is a very poor road which cannot afford two passenger trains per day each way. The total number of passengers carried might not suffice to fill one car, but it would probably be a far greater number than would be hauled if there were only one train per day. The criterion for an increase in number would appear to be as follows:

When it may be shown that the increase in facilities due to an additional train will so encourage traffic that the additional receipts will equal or exceed the cost of the additional train (which will be less than the average cost per train mile—as will be shown later) then the added train will evidently be justified.

The average revenue per passenger train mile for 1902 was given as $1.08, which includes receipts from mail and express as well as passenger receipts. The average receipts per freight train mile was $2.27, which is more than twice as much. And this is in spite of the fact that a passenger, weighing perhaps 150 pounds, paid about two cents per mile, while a ton of freight paid about 0.75 cent per mile. This great difference is partly due to the fact that the ratio of dead load to live load in freight work is about one-half, but on passenger trains it may be five or even ten. Another reason is that freight trains are made up if possible so that each engine is hauling about the limiting number of cars that it can handle (so as to reduce the number of trains required) while, as stated above, passenger trains are run frequently and light, so as to encourage the passenger traffic.

162. Monopoly in Railroad Business. One danger to be considered in the estimation of gross revenue, and also in the subsequent design of the road and the facilities offered for traffic, is the assumption that the road "will have all the traffic there is." Even ignoring the effect of possible future competition, which may be encouraged and somewhat developed by a marked lack of facilities on an existing road, it should be recognized that a large part of the traffic depends directly on the facilities offered A factory's very existence depends on its ability to collect its raw material, manufacture it and deliver it at the door of the average consumer, perhaps in a distant city, as cheaply as other manufacturers of the same article. Under close competition, an increase in one single item of expense, such as cartage from the factory to the railroad, may make up the difference between profit and loss.

The ideal location for a railroad is that it shall pass through the heart of the manufacturing district of any city and that its passenger station shall be located in the immediate neighborhood of the business center of the city. The purchase of such property for tracks and stations after the city is well established is of course very expensive, but the disadvantages of a location which is considerably removed from the ideal location are very great. These disadvantages are so increased under competition that a road's traffic may be practically ruined. Even the passenger business is greatly affected. The passengers who will travel anyway regardless of inconveniences are comparatively few.

The most important practical feature of this question lies in the fact (referred to before) that the margin between profit and loss is very small, that a very large proportion of the gross revenue must be paid out for operating expenses, that nearly all if not quite all of the remainder goes to pay interest on the bonds and that only a very small and doubtful percentage remains for dividends. It may therefore be said that the dividends come literally from the unnecessary traffic which must be coaxed and which will not travel on a road which lacks conveniences.

The force of this may be seen still more by considering the easy financial condition of a well-established road. The receipts are large and are partly spent in creating still further conveniences, commodious and convenient stations, better rolling stock, etc. These in turn encourage more traffic, which still further increases receipts, until there seems to be no end to the financial ability of the road. Such roads are the Pennsylvania, the New York Central and some others. On the other hand, the poverty of a road begets a poverty of service which still further decreases receipts until ruin is in sight. Many a road has been practically compelled to supply free cartage for freight (or allow for it by a rebate) to compensate for an inconvenient freight station. The examples of roads which have paid enormous sums to bring their passenger terminals into the heart of a great city is very instructive in this respect.

163. Division of Gross Revenue. Of the more than 2,000 railroad corporations listed by the Interstate Commerce Commission, a very large number of them are so merged with the corporations operating them that their separate existence is only evident on paper. The capital stock of many of them is partially or entirely owned by the operating company and they are operated under a great variety of leases, etc. It is therefore difficult to obtain from the financial statement of any of the large corporations a typical case of the division of gross revenue. The following case may be considered as fairly typical:

It is an independent road 371 miles long, with a capital stock of $1,114,400 and a funded debt of $9,415,000, which is made up of bonds to the amount of $8,555,000 and "equipment trust obligations" to the amount of $860,000. This is evidently a case of a road built chiefly on the proceeds of the bonds, the issue of stock being quite small. The gross revenue for 1901-2 was $1,708,937. Of this $1,101,884 or 64.5 per cent was spent in operating expenses. Of the remainder $552,821 or 32.4 per cent was needed for the "fixed charges." This left only $54,232 available for anything else. Although this amounted to nearly 5 per cent on the rather small issue of capital stock, no dividend was declared. It was evidently preferred to add this amount to their working capital or perhaps to use it in improvements. Such an action is virtually the re-investment of profits for the improvement of the road.

A similar *average* statement for the roads of the U. S. may be obtained from the I. C. C. report in which all the roads of the country are considered as one system by eliminating all intercorporate payments. The gross income, which includes a small percentage of "clear income from investments" was $1,769,447,408. Of this amount $1,116,775,785 or 63.1 per cent was spent in "operating expenses," which included a small amount for "salaries and maintenance of leased lines." The total fixed charges were $322,478,387 or 18.2 per cent; the "net" dividends disbursed were $157,215,380, which is 8.9 per cent of the gross income, but only 3.6 per cent on the $4,314,055,951 of railroad stock "not owned by railway comporations." The remaining 9.8 per cent, or $172,977,856, is the amount "available for adjustments and improvements." This includes nearly $35,000,000 spent in "permanent improvements" and some "advances to cover deficits in the operations of some weak lines;" more than one-half is set down as "surplus," just as in the case of the individual road first mentioned.

164. Fixed Charges. The fixed charges of a simple railway corporation which operates only the line which it owns will consist chiefly of the interest on its bonds. Besides this there may be the interest on "equipment trust obligations" which are merely a particular form of bond issued to pay for equipment needed by the road. Another item will be the interest on sundry interest-bearing current liabilities; this is generally but a small percentage of the fixed charges, but the current liabilities are often made to disappear by a new issue of bonds which take up an old issue and at the same cover all floating liabilities.

Other items are taxes and "permanent improvements charged to income account." But the complicated financial relations existing between operating roads and their leased lines introduces some other items which are entered under fixed charges. One of these items, which is always less than one per cent of the total fixed charges, is called "salaries and maintenance of organization." These refer to the salaries which are paid to a few of the general officers of a leased road who are retained to continue such work. Another item is placed among the fixed charges when it occurs; this is the rental paid for a leased road. As this is an "inter-corporate" payment, it did not appear in the above general summary for the roads of the United States, nor did it appear in the detailed statement of the road previously described since that road had no leased lines.

165. Net Revenue. The net revenue is that which remains after the operating expenses and fixed charges have been paid. In general it is available for dividends, but practically a very considerable proportion of it will be devoted to improvements or to the accumulation of a surplus which will serve as "working capital." During the year 1901–2 44.6 per cent of railroad stock paid *no* dividends, although the case quoted above is but one of many in which there was a considerable surplus after paying the operating expenses and fixed charges. Over 8 per cent of the stock paid dividends ranging from one to less than four per cent.

This small proportion shows the tendency to pass the dividend unless it may be made larger. Thirty-six and five-tenths per cent of the stock paid dividends varying from four to seven per cent. This represents the bulk of the stock paying normal dividends. Smaller percentages of the stock paid higher rates. One and thirty-seven hundredths per cent of the stock (amounting to $82,594,375) paid dividends of 10 per cent and over. Of course this last represents roads which are short and very exceptional in character. It should also be kept in mind that the percentages of dividend-paying stock quoted above are the most favorable of any in the previous history of railroading. If the general prosperity should ever return to the conditions existing during 1893 and earlier, when about two-thirds of all the stock paid no dividends, railroad stock would be less attractive than now in spite of the abnormal profits which are occasionally realized.

166. Operating Expenses. Uniformity per Train Mile. The classification of operating expenses here adopted will follow, both in general and in detail, the classification used by the Interstate Commerce Commission. The figures given will, in general, be *averages*. This is further justified by the very remarkable fact that the expenses *per train mile* are nearly constant, whether the trains be few or many, heavy or light. Of course there are very numerous exceptions, but it will generally be found that the marked exceptions apply to very short roads having abnormal traffic or peculiar financial relations with a parent company which is operating it.

The report for 1901–2 shows that the ten greatest railroads of the country, each operating more than 4,000 miles of road, spent $1.167 per train mile. The average for the whole United States was $1.1796. It should also be noted that the ratio of total operating expenses to total receipts from operation were 59.78 per cent for the ten roads and 64.66 per cent for the whole United States. To judge of the operating expenses of smaller roads, the figures for ten were taken from the report, the selections being made at random except that the lengths were all less than 100 miles and all of the roads were "operating roads independent." The figures are as follows:

No	Length Miles.	Operating expenses per train mile.	Ratio of total operating expenses to total receipts from operation.
1	21.25	$0.70621	71.62 per cent
2	32.60	0.47828	64.21
3	31.00	0.60649	96.12
4	64.10	0.90588	43.41
5	42.00	0.54323	63.07
6	61.00	0.75357	81.05
7	50.00	0.87456	90.32
8	50.39	2.07044	97.58
9	70.78	1.02854	53.46
10	52.20	1.74952	62.15
Average -		- $0.97167	72.30
Whole U. S.		- 1.17960	64.66

A little study of the above figures will show (as might be expected) that local conditions will so affect a very small road that its operating expenses per train mile may be considerably greater or considerably less than the average. The average value for the ten short roads here chosen is less than the average for the United States, and although two of the ten are much greater than the average, it will be found that the average value for short roads will be a *little less* rather than greater.

The reasons for the uniformity are not difficult to understand. Although the gross expense of any one item (such as rail renewals) for a large road is enormously greater than the same item for a small road, the divisor (the number of trains) is correspondingly greater and the quotient, which is the expense for that item per train mile is substantially uniform. The reasons for this uniformity will be still more apparent as the different items of expense are studied in detail. Therefore, in any future calculations of operating expenses, an average value for any item may be considered as applicable to any road, long or short, with light or heavy traffic, unless that road is subject to some abnormal operating conditions.

167. Classification of Operating Expenses. The study of the operating expenses must include not only the average cost per train mile, as shown in the accompanying tabular form, but also the percentage of each class to the total, as is shown in Table XIV herewith. In studying the tendency of any item of expense to increase (or to decrease) it must be remembered that the quantities given in Table XIV are percentages, and that, although any one item may show a lower percentage of cost than it did for previous years, the average amount spent on it per train mile *may* be greater because that percentage of the higher total cost per mile may be greater. A study of the percentages of the four general subdivisions (items 51 to 57) shows that there has been an almost regular increase in the expenditures for maintenance of way for many years past. This has been partly to make up for the excessive economies prac-

Year.	Operating expenses per train mile.
1892	$0.96580
93	.97272
94	.83478
95	.91829
96	.93838
97	.92918
98	.95635
99	.98390
1900	1.07288
01	1.12292
02	1.17960
Average,	.99680

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tised during the panic times of 1893, and also because the railroads are taking advantage of the recent prosperous years to place the roadbed in the best possible condition in anticipation of the slump which may follow.

Maintenance of equipment has followed almost exactly the same course—an increase not only in percentage, but a very large increase in the gross amount spent, and for the same reason as before. The expenses of "conducting transportation" are not so controllable as those of maintenance. They depend more closely on the volume of business done. Although the percentages have been decreasing on account of the enormous increase in maintenance charges, the actual expenditures have increased on account of the increase in business done.

The "general expenses" show a reduction of 20 per cent (in percentage) from 1895 to 1902, but by multiplying the percentages given by the cost per train mile for those years, it will be seen that even the expense per train mile has increased. This indicates the general increase in the scale and cost of living in this country.

168. Maintenance of Way and Structures. Repairs of Roadway. The work involved under this item has been referred to before under track work. The item includes all the items of keeping up the roadway which are not specifically included under the succeeding items. Its gross amount is enormous and even its percentage has been increasing almost steadily for many years past. About 60 per cent of the cost of track maintenance is due to labor alone. The average wages paid to trackmen has increased from $1.17 in 1895 to $1.25 in 1902. The fluctuations in the rates paid to trackmen seem to follow very closely the fluctuations in this item.

Renewals of rails. This item includes the cost of the rails, their inspection, and their delivery to the road, but it does not include their local distribution nor the cost of laying in the track, which is charged up to "repairs of roadway." The fluctuations are due partly to fluctuations in the price and partly to the fact that rail renewals may be somewhat staved off during hard times until better times come, or until the roadbed gets in such a condition that renewals are imperative and then they come in a lump.

Renewal of ties. The fluctuations in the percentages for this item are irregular and not readily explicable. The increasing attention which is being paid to ties and their preservation on account of their growing scarcity may be somewhat responsible for some of the fluctuations. As with the previous item, the cost of distributing and laying is placed under Item 1, but the item does include the first cost, inspection and delivery to the road and also the cost of tie plates, tie plugs and chemical treatment, if such is used.

Items 4 to 10. (Table XIV) The present object of the study of these items is the determination of the effect of such changes of alignment as an engineer may make on these items in the maintenance of a road. Items 4 and 6 are large items, but the changes of alignment which will be hereafter discussed will not ordinarily affect these items unless it changes them proportionately to the change in alignment. Any such effects will be noted under their proper heads. The other items are so insignificant that any ordinary error in estimating their effect cannot appreciably influence the final result.

169. Maintenance of Equipment. *Item 11. Superintendence.* This item includes the salaries of superintendent of motive power, master mechanic, etc., and therefore is independent of small fluctuations in business done or in condition at length of line.

Item 12. Repairs and renewals of locomotives. The cost of this item is of importance to the locating engineer for he must know the effect on this item of a change in distance. It may be said at once that the cost of this item is not directly proportional to distance and the same remark applies to many other items. Curvature affects the cost of repairing very greatly—chiefly in wheel wear, and the engineer must consider this in estimating the value of a saving in curvature. The rate of grade also has an effect on this item. The item includes all those expenses whose effect is supposed to last for an indefinite period in contradistinction to the daily work of cleaning boilers, packing cylinders, etc., which is done by roundhouse men and which is charged to item 21.

During the first years of the life of a locomotive, the repairs (barring accidents) will be small, but as the locomotive grows older they increase in a growing ratio. When the annual repair charge becomes one-fourth (or in exceptional cases one-third) of its first cost, the locomotive should be sent to the scrap pile, for in such cases the cost per train mile becomes larger than a reasonable annual charge, allowing for all depreciation, on the cost of a new locomotive. When an old locomotive is replaced by one of a better and more costly type, the *excess* cost should be charged to betterments or "permanent additions to equipment."

The great increase in the percentage of this item and the still greater increase in gross cost has been due to the unprecedented increase in volume of business, which has required the utilization of every locomotive capable of hauling a load. If the immediate future should see a marked slump in the business done by railroads, there would probably be a reduction in this item. The effect of various changes of alignment on this item will be considered under their several headings.

Items 13, 14 and 15. Repairs and renewals of cars. As in the previous item, there is included all expenses necessary to keep the cars up to the full standard as to number and condition, but it excludes all consumable supplies (oil, ice, waste, etc.) and also any excess of cost due to renewals being of higher grade and cost than the cars replaced. The engineer is chiefly concerned with the cost of wheel repairs (affected chiefly by curvature) and drawbar repairs, which are largely affected by grade.

Items 16 to 19. These items are very small, even in the aggregate. Under ordinary conditions, no change which an engineer may make will affect these items in any appreciable amount.

170. Conducting Transportation. *Item 20. Superintendence.* As before, this item is not affected by any small changes in alignment.

Item 21. Engine and roundhouse men. This is one of the largest single items in the whole list. It includes the wages of all enginemen and such of the roundhouse men whose work is in the daily care of the engines in contradistinction to those employed in making repairs (item 12). There has been a steady increase in the daily wages of such men since 1894. Trainmen are usually paid on a mileage basis. This will not of course be affected by changes in curvature and grade but changes in distance may affect train wages to its full proportionate value. This will be discussed in detail under "Distance."

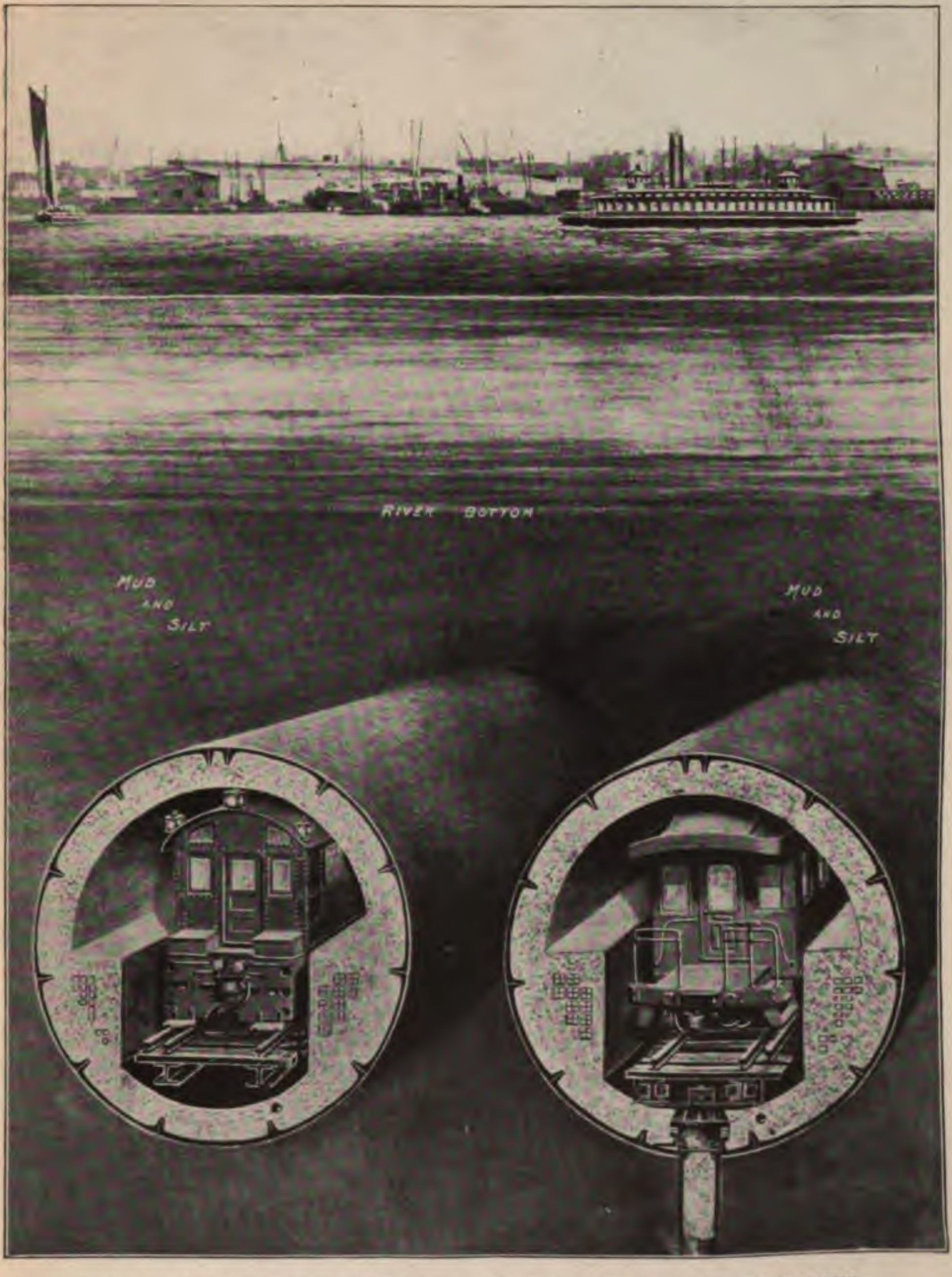
Item 22. Fuel. With the exception of Item 1, Repairs of roadway, this is the largest single item in the list. And it has grown, even in percentage, during recent years, owing to the general advance in wages and in the scale of living. The item includes the total cost of the coal in the engine tender and therefore includes the cost of delivery, distribution, and even the cost of operating coaling stations. The amount of fuel consumed is quite largely affected by changes in alignment, although it will be shown that an increase in distance will not add its proportionate amount to the consumption. The absolute cost per ton is exceedingly variable. Roads which run through a coal region can sometimes obtain their coal for less than one dollar per ton, while roads which are far removed from the coal regions may have to pay six or eight dollars per ton.

Items 23, 24 and 25. Other locomotive supplies. Although there are some fluctuations in these items there is no apparent tendency toward any regular increase or decrease. It is usual to consider that changes in alignment will affect these items in the same proportion as they affect the use of fuel. Although such an assumption is not strictly correct, the total of the items is so small that any error in the assumption is of no practical importance.

Item 26. Train service. This large item includes the wages of all trainmen except the engine crew. As in item 21 there has been an almost regular increase in rate of compensation per day (although the *percentage* of the item per train mile has decreased) since 1894. And also, since the trainmen are paid on a mileage basis, it will affect the cost of operating any considerable changes of distance.

Item 27. Train supplies and expenses. The item includes all that large list of consumable supplies which are used on the train but not on the engine, such as ice, illuminating gas, oil for lamps and for lubrication, coal used in car stoves, etc. The consumption of some of these articles is chiefly a matter of time; of others, it is a matter of mileage. Fortunately the whole item is small, about $1\frac{1}{2}$ per cent, and the uncertainties, as affected by changes in alignment, are insignificant.

Items 28 to 32. Although some of these items are large and in the aggregate make up about 13 per cent of the total, they are but little if any affected by any changes which an engineer is liable to make. There has been an almost uniform decrease in



TWIN RAILWAY TUNNEL TUBES UNDER THE HUDSON RIVER AT NEW YORK

Built for the Pennsylvania Railroad. Tubes of steel and concrete.

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the percentage of these items per train mile during the last ten years.

Item 33. Car mileage. To save the rehandling of freight at the junction points of railroads, it has become the universal practice that freight in carload lots (or even in less than carload lots) shall be transported from shipping point to destination in one car, no matter how many roads it may pass over. Each road receives its proportionate amount of the total freight earnings received for the shipment, but it also pays to the road owning the car (which *may* be a road having no part in that particular shipment) a mileage rate which is supposed to represent the value of the use of the car. Cars which are sent away from the home road are supposed to be returned to the home road at the first opportunity. Any accident which may happen to a car while on any road must be made good by the road on which it occurs. This wise agreement among roads compels each road to place inspectors at all junction points who inspect all arriving cars and who refuse to accept any car for transportation which does not conform with rigid rules regarding its physical condition.

By a sort of clearing house arrangement the roads settle their debit and credit accounts with each other by the payment of a cash balance. The item represents the amount of this balance and really shows the amount paid in cash by a road for cars in which to handle business on which they receive earnings. In that respect it represents hire of equipment which is hired to supply their own deficiencies. Since the payment for such cars is strictly on a mileage basis, any change in distance must be allowed for at its exact value. During recent years railroads have come to an agreement providing for a per diem charge in addition to the mileage charge to compensate for unreasonable delay, but such bookkeeping is of no concern to the locating engineer. The percentage of this item has been growing steadily less during recent years.

Item 34. Hire of equipment. This very small item represents the payments made by some roads for special equipment or that paid by a few roads which are so small and poor that they hire some or all of their rolling stock. The item is non-existent in the accounts of a majority of roads, and in any case it is of no concern to the locating engineer.

Items 35 to 37. Damages, wrecks. These items are of no concern to the engineer except as he may consider the danger of accidents to be affected by curvature. This element will be discussed under Curvature.

Items 38 to 53. Under ordinary conditions these items are not affected by any changes which an engineer can make. They consist chiefly of those general expenses which are unaffected by modifications in location. They need not be further discussed here.

ECONOMIC LOCATION.

171. General Principles Involved. A hasty mental review of the previous discussion, as well as a few considerations of common sense, will show the truth of the following statements:

(*a*) Disregarding the comparatively rare cases in this country where a practicable location of any kind is a creditable engineering feat, it may be said that a comparatively low order of engineering talent will suffice to lay out a line along any general route over which it is physically possible to run trains, and that there are usually several such possible routes. The route selected may not be favorably located for obtaining business, its alignment may be such that its operating expenses are high, and the ruling grades may be so high that only light trains can be run, but the road *can* be operated.

(*b*) Among the many possible routes which may be selected for a road, there is one which is superior to any other from an operating or business standpoint, and it is the province and test of the engineer to select that best route.

(*c*) There are several more or less conflicting interests which must be studied—(1) the maximum of business must be obtained, but this is sometimes only obtainable at great initial cost; (2) the ruling grades must be made as low as possible, which is generally costly, and it *may* require a location which will sacrifice some business; (3) the alignment must be kept easy so as to reduce operating expenses, but this usually is very costly; (4) the total cost must be kept within a figure which will be justified by the future earnings and also leave enough margin as working capital out of the total funds which are raised, so that the road may continue to operate during the five or ten years which is required to build up the "normal" traffic.

(*d*) Each new route suggested forms a new combination of the above conflicting elements, and the business of the engineer is to estimate and compare these elements, selecting the combination which will give the largest return for the least outlay, considering both initial cost and future operating expenses as elements of the outlay. As a basis for this work, the engineer must understand the effect on operating expenses of changes in distance, curvature, and grades. The normal subdivision of operating expenses has already been considered. It therefore remains to compute the effect on several items of changes in alignment.

172. Reliability and Value of Economic Calculations. The student should not form the idea that the following calculations will enable one to compute with mathematical precision the effect of changes of alignment. There are far too many elements involved, and the effect of certain influences is variable. But although a precise solution is unobtainable, a solution which is sufficiently accurate for practical purposes may be made, which is infinitely better than no solution at all. For example, suppose that a very crooked stretch of road may be changed to comparatively easy alignment which saves considerable curvature by an additional expenditure of say $2,000. Assume that there are 20 trains per day over that line, and that it has been computed (by methods developed later) that the operating expenses would be reduced by $350 per year by the reduction of that curvature. As $350 per year, capitalized at 5 per cent, is equivalent to an investment of $7,000, and the improvement may be made for $2,000, the latter is evidently justifiable. Such is the bare outline of the method.

The estimate of the cost of the improvement may be accurately made, but it is not claimed that the estimate of the saving per year is precise. It may, however, be shown that, even with ample allowances for the uncertain items, it is practicable to assign upper and lower limits between which the truth must lie. A greater knowledge of the subject and greater experience on the part of the engineer will enable him to narrow those limits so that the error is immaterial. And frequently even this is unnecessary. The real question is not whether the capitalized value of the improvement is $7,000 or $5,000 or $9,000. It may be that an improvement which would make possible that saving may be made for a few hundred dollars, or it might require $20,000. In either case, the true answer is unquestionable.

If the cost of the improvement is very nearly equal to its computed capitalized value, then no great harm can come from either decision, for the decision would then be based on the willingness of the company to spend additional money. The method furnishes a criterion, which even in the hands of an inexperienced engineer has some value, and which alone gives value to his opinion. But the method enables the experienced engineer to give the best opinion which is obtainable, for it enables him to apply his experience to a method of computation which approaches accuracy as nearly as may be.

It must not be supposed that the numerical values worked out in the following pages are necessarily applicable to any assumed case. They are given to show the *method* of their derivation, and should be modified to fit local conditions according to the best judgment of the engineer.

DISTANCE.

173. Relation of Distance to Rates and Expenses. Rates are usually based on distance traveled on the apparent assumption that the value of the service rendered and the cost to the company are directly proportional to the number of miles traveled. Neither assumption is true. If a passenger or a load of freight is to be transported from one city to another city 100 miles away, the service rendered is to accomplish the transfer as easily and quickly as possible. If another road were constructed, perhaps at extravagant cost, by which the distance were cut down to 90 miles, that road would render a greater and better service, because it would reduce the necessary travel, and yet on the mileage basis the shorter road would be entitled to less than the other in spite of the fact that it renders a better service.

The assumption that the cost is proportional to the distance is more nearly correct, although, as will be shown later, even this is far from accurate. It is not difficult to compute an average cost for a large number of passenger trains, and by dividing it by the total passenger mileage, obtain a value of the cost of a "passenger mile." But the additional cost of transporting one additional passenger on a regular train is hardly more than the cost of printing his ticket. Even if it were practicable to compute the extra consumption of coal and the infinitesimal addition to other operating expenses due to his being on the train, the added cost would evidently be but an insignificant fraction of the average cost of a passenger train mile. The same argument holds if we consider the additional cost of an extra ton of freight.

By the same line of argument it will be shown that a change in distance will not affect the cost of running trains in proportion to the change. It is easy to see that general expenses will be absolutely unaffected by an alteration of alignment which saves a mile in distance, and it will be shown that even the consumption of fuel does not vary in proportion to the distance. If it were practicable to construct a tariff of rates which should consider excessive curvature and grades on the various parts of the line and make the rates dependent on them as well as on many other constructive features which add to the cost of operation, the rates would be more nearly proportional to the cost, but the public would not appreciate it and it would be useless work. And when it is further shown that it is sometimes justifiable for a road to haul competitive business at a rate actually less than the average cost of their traffic, it will be seen that the relation of distance to rates and expenses cannot be expressed by any simple proportion.

174. Effect on Operating Expenses. Studying the items of Table XIV, it may be seen first that the cost of items 1, 2, 3, and 5 for any additional length of track will evidently be proportional to the distance. These items make up nearly three-fourths of the total cost of maintenance of way. The effect of the other items will depend largely on the extent of the distance added, whether a few hundred feet or measured in miles. In any particular case, if there is any radical difference between the lines compared, the effect of the particular change (as for instance the cost of maintenance and operation of a drawbridge divided by the number of trains using it) should be added to the cost of operating that route.

Items 4 and 6 must be studied for each problem in this respect, but for average figures we may say that when the distance is great we may add 100 per cent, and when it is small, we may neglect them altogether. The same estimate may be made for items 9 and 10. Except in special cases, a change of plans will not affect item 7, and even if it did it would be a separate affair and form no part of the problem of a change in distance. It is estimated that item 8 will be increased about 60 per cent by an increase in distance. Fortunately the item is so small that even a large change in this estimate will not appreciably affect the final result.

The effect of an increase in distance on items 11, 17, 18, and 19 is very uncertain. From one standpoint it is difficult to see how there would be any effect, especially if the change is small. But from the broad standpoint that any increase in distance must have its effect even though it is not directly perceptible 50 per cent has been added for small changes of distance and 100 per cent for greater changes. As before, the insignificance of these items will prevent any probable error in these estimates from being appreciable. Item 16 will not be affected.

The larger items 12 to 15 need special study. Rolling stock deteriorates (1) with age, regardless of usage; (2) on account of strains due to stopping and starting; (3) on account of wear and tear due to curvature; (4) on account of the extra strains due to grades; (5) on account of strains due to traction on a straight and level track or to that part of the traction for which curves, grades, and other such causes are not responsible, and (6) the deterioration of locomotives is largely due to the strains of expansion and contraction, especially of the firebox, when fires are drawn.

Note that of all the above causes, the mere addition of distance is responsible for one, and for one which is evidently but a minor part of the total. If the added distance involves added curvature and grade, these must be computed separately. We must first estimate the effect of cause (5) on item 12. The records of engine repairs show the proportionate cost of repairs to boiler, repairs to running gear, etc. The method and result here given is that developed by the late A. M. Wellington several years ago, and in the absence of anything better it will be used.

The boiler is found to be responsible for 20 per cent of the repairs to engines, and he divided the 20 per cent by assigning 7 per cent (about one-third) to "terminal service, getting up steam and making up trains," 4 per cent to curvature and grades, 2 per cent to starting and stopping at way stations, and the remaining 7 per cent to distance on tangent between stations. The other items of engine repairs are similarly treated with the net result that mere distance is estimated to be responsible for 42 per cent of all locomotive repairs, and this estimate will be used for all changes in distance, great or small. The causes of deterioration of cars may be classified as above, omitting merely the last.

A very large part of the repairs of freight cars are those due to draft gear, the injuries to which result almost exclusively from the hard service they receive in terminal yards and on the ruling grades of the road. Another very large item is the renewal of car wheels, the wear of which is due very largely to curves and to the action of brakes on heavy grades. In view of all these facts it is not surprising that Wellington allowed only 36 per cent of the repairs as being due to mere distance. This applies to items 14 and 15.

The repairs of passenger cars are somewhat different. They are not subject to such rough usage as freight cars, but we may assume that the proportion of repairs of their drawbar gear, which is due to mere distance, is the same as with freight cars. The effect of mere distance on running gear repairs may likewise be considered the same as with freight cars. A very large item in the repairs of passenger cars is for painting and varnishing and such deterioration is due largely to mere time and age. Another item is for the car furnishings whose deterioration depends on a variety of causes. Although many of these sub-items should have something more or less than 36 per cent assigned to distance, that figure will also be used for item 14. Even admitting an error of 20 per cent in this estimate, it would affect the final result by less than one-half of one per cent.

Among the items of conducting transportation, we may at once strike out items 20, 30, 31, 32, 34, and 38 to 46 as being unaffected by any changes of distance, great or small. We may also say from mere inspection that items 33, 35, 36, and 37 should be allowed the full 100 per cent.

Item 21. As previously intimated in § 170, train wages are paid on a mileage basis usually, but it is a basis which is not strictly proportional to the mileage. When practicable and convenient, a "division" of a road is made nearly 100 miles long, and this constitutes a "run." When the division is longer than this, it may be called 1⅛, 1¼ runs, etc. The trainmen are therefore paid according to the number of runs made per month. This is especially necessary in freight service in which there is often no distinction of day or night, weekday or Sunday, and when any method of compensation based on the number of "days" of work becomes impracticable.

The practical effect of this is that any slight changes in distance, measured in feet, would not change the calculation of wages by the smallest unit of its measurement, but when the change must be measured in miles, we must give it its full value. Item 21 includes the wages of roundhouse men which constitute about 7 per cent of the item, and of course their wages are unaffected by any change of line. Therefore we may say that 93 per cent of item 21 will be affected by changes measured in miles, but that the item is unaffected by small changes in distance. The same line of argument applies to item 26 except that 100 per cent is allowed for large changes and nothing for small changes.

Item 22. Fuel. The causes affecting the consumption of fuel may be classified as follows:

(1) Mere firing up, drawing of fire and banking, which has been estimated to be responsible for 5 to 10 per cent of all the fuel consumed, for the policy of banking fires consumes fuel although it saves the locomotive by avoiding so many excessive changes in temperature.

(2) Consumption while standing still, which is supposed to amount to from 3 to 6 per cent of the total, for the time spent standing still while steam pressure is up amounts to a considerable proportion of the total time, especially with freight engines.

(3) Stopping and starting, especially the latter, for the tractive resistance is then very great, the engine works at full stroke,

making a heavy draft, forcing the fire; this is estimated to consume 10 to 20 per cent of the total; and in the case of trains making frequent stops and high speed between stops, such energy forms a large part of the total.

(4) Curvature is estimated to be responsible for about 3 per cent of the total; such a value is only an average value at its best, and does not mean that 3 per cent of the consumption for any one mile, or even for any one division, would necessarily be required to overcome the curvature resistance, but it attempts to give an average value based on the average curvature per mile of road in the country and on the added resistance caused by such curvature.

(5) Grade, which is computed, by a process similar to that above, to be responsible for 15 per cent of the total as an average value. Adding up these percentages, we find a minimum of 36 per cent and a maximum of 54 per cent, or an average of 45 per cent, as the percentage of fuel which is *not* required for direct hauling. This shows that only 55 per cent of the average consumption per mile is required for an addition of one mile of straight and level track. If the added track has curvature and grade, an extra allowance must be made for that, as will be developed later.

Items 23, 24, and 25. When the water supply comes from the company's own pumping stations, the additional supply for an additional mile is practically insignificant; if it comes from a metered municipal supply, its additional cost will be nearly proportional to the added distance; therefore, as an average value 50 per cent will be added, and the same amount will be added for items 24 and 25.

Item 27, train supplies, includes items which are evidently directly affected by changes in distance and others which are but little if at all affected; therefore an average of 50 per cent will be allowed.

Item 28. The necessity for flagmen and watchmen must be considered as being directly proportional to the distance, but on the other hand an addition to distance will not usually increase the number of switchmen who are chiefly required in yards and terminals; therefore 50 per cent will be added.

Item 29 includes the wages of operators (unaffected) and the expenses of running the stations, maintenance being provided for in item 8; although a little more battery material would be expended in sending the current through the additional line, the increase in expense will be so utterly insignificant that it will be ignored.

Collecting the above estimates in a tabulated form we have Table XV. This shows that if the additional distance is small, the cost per mile of running a train over the extra distance will be only 36 per cent of the average cost of a train mile and that the cost is increased to 58 per cent if the added distance is very great. From 1895 to 1902 the average cost of a train mile increased from less than 92 cents to nearly $1.18. The average from 1892 to 1902 was 99.68 cents. Although the present high value will probably be somewhat reduced in the future, it is unlikely to sink to

TABLE XV.

Effect on Operating Expenses of Great (and Small) Changes in Distance.

Item No.	Normal average.	Per cent affected.		Cost per mile.	
		Great	Small	Great	Small
1	10.779	100	100	10.78	10.78
2	1.442	100	100	1.44	1.44
3	3.060	100	100	3.06	3.06
4	2.490	100	0	2.49	0
5	.557	100	100	.56	.56
6	2.096	100	0	2.10	0
7	.256	0	0	0	0
8	.145	60	60	.09	.09
9	.038	100	0	.03	0
10	.352	100	0	.35	0
	21.205	......	......	20.90	15.93
11	.631	100	50	.63	.32
12	6.258	42	42	2.63	2.63
13	2.218	36	36	.80	.80
14	7.048	36	36	2.54	2.54
15	.188	36	36	.07	.07
16	.218	0	0	0	0
17	.538	100	50	.54	.27
18	.041	100	50	.04	.02
19	.503	100	50	.50	.25
	17.643	......	......	7.75	6.90
20	1.759	0	0	0	0
21	9.640	93	0	8.97	0
22	9.949	55	55	5.47	5.47
23	.649	50	50	.32	.32
24	.372	50	50	.19	.19
25	.192	50	50	.10	.10
26	7.450	100	0	7.45	0
27	1.510	50	50	.75	.75
	31.521	......	......	23.25	6.83
	31.521	......	......	23.25	6.83
28	4.057	50	50	2.03	2.03
29	1.906	0	0	0	0
30	7.497	0	0	0	0
31	.730	0	0	0	0
32	.331	0	0	0	0
33	1.921	100	100	1.92	1.92
34	.279	0	0	0	0
35	.783	100	100	.78	.78
36	.910	100	100	.91	.91
37	.155	100	100	.15	.15
38	.860	0	0	0	0
39	.427				
40	1.647				
41	.146				
42	.107				
43	1.776				
44	.476				
45	.620				
46	.575				
	56.724	......	......	29.04	12.62
47	1.133	0	0	0	0
48	1.326				
49	.284				
50	.410				
51	.680				
52	.168				
53	.427				
	4.428	......	......	0	0
	100.000	......	......	57.69	35.45

the low value of 1895 unless there is some radical change in methods of operation.

Calling the cost of operation per train mile the average value of $1, the above percentages may be at once considered as *cents* per train mile. A regular train running each way every day of the year would cost 2 × 365 times the above values. If the train only ran on week days we would need to use 313 instead of 365. The added annual cost per daily train each way for each *foot* of distance is

$$\frac{35.45 \times 365 \times 2}{5280} = 4.90 \text{ cents} \qquad \mathbf{(104)}$$

Similarly the added annual cost per daily train each way for each *mile* of added distance is

$$57.69 \times 365 \times 2 = \$421.14 \qquad \mathbf{(105)}$$

175. Numerical Example. *Reduction of distance.* Assume that by straightening the line of a road through a rough section of country, although at the expense of considerable earthwork, the line may be shortened 342 feet. Assume that there are 16 trains per day each way. How much expenditure will be justifiable to accomplish this? Using the value given in equation 104, we have an annual saving of 342 × 16 × 4.9 cents = $268.13. Capitalizing this at 5 per cent it would justify an immediate expenditure of $5,362.

In order to give some idea of the reliability of this estimate, and the effect of a very large change in some of the items in Table XV, the estimation of which may be considered doubtful, assume that the estimates of 42 and 36 per cent of the normal average for repairs of rolling stock is considered too small and it is made *double*—or 84 and 72 per cent respectively. Suppose also that the item for fuel is made 75 per cent instead of 55 per cent. The combined effect of these additions would be to compute the saving per foot to be 6.01 cents instead of 4.90 cents, while the capitalized value would be increased to $6,577 instead of $5,362. The additions to these items were purposely made excessive and beyond any reasonable estimate of their true value, and yet the final value was altered only 22 per cent.

As said before, the value of these estimates lies in having some criterion, even though inaccurate, rather than none at all. If the suggested improvement can be made for say $2,000, it is evidently justifiable beyond question. If it will cost $20,000, it is evidently unjustifiable, equally beyond all question, and yet without some such method of computing the operating value of such an improvement, few engineers could form much idea of its value.

176. Effect on Receipts. Among all the details of alignment, distance is the one for which there is some compensation in an increase, and that is because rates are based on distance rather than on curvature or grades. Although it is unquestionably contrary to public policy to burden traffic unnecessarily by an increase in distance, yet it may be demonstrated that the added receipts from non-competitive traffic due to such increased distance will amount to more than their added cost. But in order to study this feature properly the distinction between competitive and non-competitive rates must be noted. For our purposes traffic may be classified as "through" and "local," in which through traffic refers to that which passes over two or more roads, no matter how long or short any section of the trip may be, and in which local traffic refers to that which is confined to one railroad system though it may run from one end to the other. Further subdivison is necessary as follows:

A. Non-competitive local—on one road with no choice of routes.

B. Non-competitive through— on two (or more) roads but with no choice.

C. Competitive local—a choice of two or more routes, but the entire run may be made on the home road.

D. Competitive through—direct competition between two or more routes, each passing over two or more lines.

E. Semi-competitive through—a non-competitive haul on the home road and a competitive haul on foreign roads.

Receipts for traffic passing over two or more lines is divided between the lines in proportion to mileage. "Terminal charges" are sometimes subtracted from the amount before the division is made and sometimes a strong road forces a weaker road to submit to some other exaction before the division is made, but the final division is made in proportion to the mileage for each passenger ticket or each freight bill. We have seen in §174 that the cost of operating an additional mile is about 58 per cent of the average cost. This means that on all non-competitive business (Class A) there is an actual profit in this added distance. On the other hand, competitive rates are made with small regard to distance, are generally equal, and therefore any added distance results in a sheer loss without any compensation. This applies to all the traffic of class C.

The other classes of traffic are affected by distance in various degrees between these two extremes. To illustrate: Suppose that the distance on the home road for any given shipment is 100 miles and the distance on the foreign road for that shipment is 150 miles; suppose that the freight charge is $10; then the home road will receive $\frac{100}{100 + 150} \times \$10 = \$4.00$. This means 4 cents per mile for that particular class and weight of freight. Suppose that the distance is increased 5 miles on the home road, but assume that the traffic is wholly competitive and therefore that the total rate received will be $10, regardless of the added distance. Then the home road will receive $\frac{105}{105 + 150} \times \$10 = \$4.1176$. If we allow to the original 100 miles its full previous allowance of 4 cents per mile, we have left 11.76 cents to pay for the extra 5 miles. This is at the rate of 2.352 cents per mile, which is 58.8 per cent of the four-cent rate.

This is slightly more than the previously computed percentage of added cost for additional distance computed in miles. Therefore, if the original four-cent rate is profitable, the added receipts due to the added distance will be sufficient to operate the added distance profitably. Incidentally, the foreign road suffers, for it will receive less for precisely the same service. The above numerical case is very nearly at the dividing line between profitable and unprofitable additions to distance. If the length of the home road is but a small proportion of the total distance, then it may be similarly computed that an addition to distance is distinctly profitable. On the other hand, if the length of the home road is a large proportion of the total distance, an addition to distance is distinctly unprofitable, and when the length of the foreign road is zero (which means that the competitive haul is entirely on the home road) then any addition to distance is sheer loss without any compensation, even partial.

The above numerical case represents but one of an almost infinite number. Each station along the line has possible traffic connection with almost every other railroad station in the country. The route from each station to every other station represents a new combination, and the net effect of the added distance is the combined effect of all the separate cases. This instantly shows that a precise mathematical solution is impossible, but the above solution has value in pointing out some general truths as follows:

(*a*) In *all* non-competitive business, whether through or local, the added receipts due to added distance will be profitable, and if the business of a road is almost entirely non-competitive there is little or no disadvantage in added distance, especially if the construction is cheapened in spite of the added distance.

(*b*) When roads handle a very large amount of competitive business any additional distance may be a source of loss on that class of business, and the loss may be so serious as to justify a considerable expenditure to reduce it. Another reason for the subsequent expenditure of money to reduce distance is that, after freight rates are once established between roads on through business, they are not apt to be disturbed to make them conform to the slight fluctuations of distance caused by changes in the alignment.

(*c*) The above statements can be reduced to the general conclusion that since every road handles a considerable proportion of non-competitive business, there is always some compensation for the added expenses of operating additional distance. The majority of small roads do a business which is almost wholly non-competitive, and to them the added receipts will usually pay for the added distance, even if it is not an actual source of profit. Finally, it may be said that a road is not usually justified in making an additional expenditure to shorten distance (*i. e.*, adopt a route which will have a greater gross cost in spite of the shortened distance) unless it handles a very large amount of highly competitive business.

There are some other considerations which must not be ignored in considering this question. One of them is the question of the additional time required to make the trip. But this is ordinarily to be considered only with respect to competitive passenger business, and even then the cases are confined to the competition of trunk lines between important cities. The other consideration is the possible effect on the business done. "A short straight line" is the popular description of a well-designed road. If the engineer's aim for a *short* road leads him to pass by sources of income so as to lose them, his road will have little business and reduced receipts because it is short. As a general rule "adopt that route which will give the greatest traffic per mile of road." On the one hand this avoids the error of running a line which is excessively crooked in the effort to secure every possible element of traffic and thus burdening the whole traffic with an excessive haul, and on the other hand running a line which misses important sources of traffic in the effort to have a straight line.

CURVATURE.

177. Operating Disadvantages of Curvature. The non-technical mind appreciates, even too readily, the disadvantages of curvature. But it is generally true that the disadvantages which are most thoroughly appreciated by the public are of least economic value to the engineer. The several disadvantages will be classified and discussed in an order which is perhaps the inverse order of their importance.

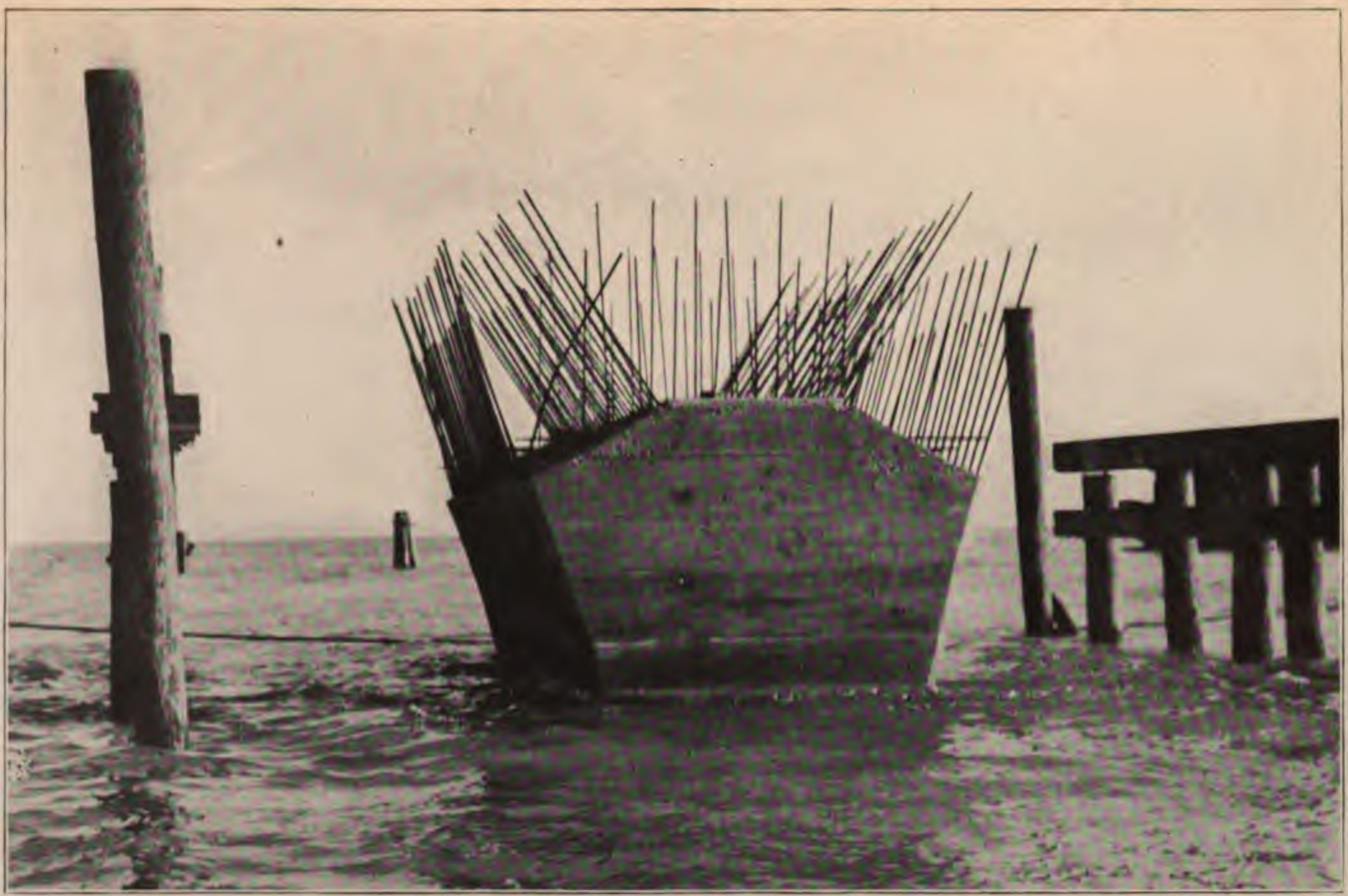
(1) It increases the danger of collision and derailment and aggravates the damages of a derailment when it occurs. The application to be made to this statement of undoubted fact is,—how much is a road justified in expending in order to reduce or eliminate any given curve? Since the entire elimination of curves is a physical as well as a financial impossibility, the question reduces to the lessening of danger from accidents that would result from such reductions as are possible. The I. C. C. report on railroad accidents for the year ending June 30, 1902, showed that the number of passengers carried one mile for one killed was 57,022,283. This means that the chances are even that a passenger could ride 57,000,000 miles before he would be killed. If he were to ride

continuously at the rate of 60 miles per hour, it would require over 9,500,000 hours or nearly 400,000 days, which is considerably over 1,000 years.

But how many of such casualties are due to curvature, and how many million miles must be traveled by the average passenger for one killed due to curvature which is physically or financially avoidable? If all of the above traveling was done on the railroads of this country, on which there are over 200,000 curves, what is the probability of its happening on any one curve and how much may be spent on that curve to reduce the danger? And if it were spent, would there remain *no* danger of an accident there? A thorough logical analysis of this question shows that although it is always proper to take reasonable precautions to avoid accident at an especially dangerous curve (such as maintaining a flagman there) it is impossible to assign any financial value to to the mere *danger* of accident which would accomplish anything toward modifying construction.

(2) Curvature may affect traffic (*a*) by reducing the possible speed of fast trains. There is some force to this objection as it applies to sharply competitive traffic between two cities—a traffic of which most roads have not a trace. The extent to which the passenger traffic might be increased by the minute or two which might be saved is however so uncertain that it defies analysis. (*b*) It may produce rough riding, and (*c*) it may create an apprehension of danger which may of itself deter travel. The disadvantages resulting from all three of these sub-causes are greatly reduced by good roadbeds and transition curves. Freight traffic, which comprises about two-thirds of the total, is wholly unaffected by it, and the passenger traffic of most roads will not be influenced by it, and therefore an engineer is not ordinarily justified in giving it any financial weight.

(3) It *may* affect the operation of trains (*a*) by limiting their length and (*b*) by limiting the type and weight of engines. There are a few instances known where roads which run along a river bank, have very easy ruling grades and on which the curvature is perhaps very sharp on account of sharp bends in the river. On such roads the curvature may be the feature which limits the length



COMPLETED CONCRETE PIER READY TO RECEIVE ARCH FORMS

The reinforcing rods protruding from the pier are continued through the body of the arch. Construction of Long Key viaduct on Key West extension of Florida East Coast Railway.

of trains, but such cases are rare and even when they occur a computation similar to that later developed will show how much may profitably be spent to reduce the rate of curvature. If a long grade up a mountain were kept uniform, regardless of curves, the curves would add such resistance that they would limit the length of trains, but good practice requires that the grades shall be "compensated for curvature," as explained later.

The excessively sharp curvature which has been used on some mountain roads will certainly preclude the use of some of the largest types of locomotives. But such roads do not have a traffic which justifies the use of the heaviest locomotives. And it has been found that even consolidation engines have an ability to run around sharp curves which is surprising and which permits their use on any ordinary curvature.

(4) Curvature increases operating expenses. This disadvantage is definite, positive and approximately computable and since a reduction in expenses may be made by reducing curvature, we must calculate the effect of curvature on operating expenses.

178. Effect on Operating Expenses. Although no exactitude may be claimed for the statement, it is nearly true that the expense of operating trains on curves between two tangents is independent of the degree of curve. If A B in Fig. 157 is a 5° curve and A′B′ is a 12° curve between the same tangents, the rail wear per foot of curve and the wheel wear per foot of travel on the curve will of course be greater on the 12° curve, but the curve is correspondingly shorter. Therefore the assumption is made that the total rail wear due to the passage of a given number of trains from A to F same as that from A′ to B′;

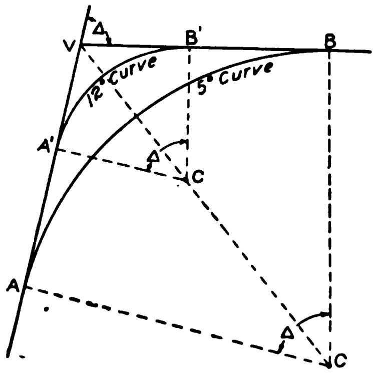


Fig. 157.

a similar assumption is made regarding other expenses which are affected by curvature, and this leads to the conclusion that the cost of operating a given curve is a function of the number of the degrees of central angle and that it is independent of the radius of curvature.

While the train resistance due to curvature depends on the velocity and on other causes which makes its relation to degree of curvature somewhat uncertain and even variable, it may be stated as an approximate figure that the added resistance due to a continuous 10° curve is about equal to that on a straight and level tangent. Doubling the track resistance will increase certain items of expense by computable amounts. Their combined value is a certain percentage of the cost of a train mile, but since a continuous 10° curve will have 528° of central angle in one mile, 1° of curvature will be responsible for $\frac{1}{528}$ of that added cost. Such is the bare outline of the method.

It will simplify the calculations while studying the items of Table XIV to throw out at once those items which will be unaffected by curvature. Beginning with the items of maintenance of way, we may at once throw out items 5 to 10. Ordinarily even item 4 may be thrown out since culverts are unaffected and bridges and trestles are not placed on curves if it can be avoided. In a special case allowance should be made for this item.

Item 1: A very large proportion of the sub-items, such as ditching, weeding, care of embankments and slopes, are unaffected; assuming that 50 per cent of the sub-items are unaffected and that the others are affected an average amount of 50 per cent of themselves, it would add 25 per cent to the cost of this item.

Item 2: Experiments on the relative wear of rails on tangents and on curves would seem to indicate that the wear on a 6° curve is about twice that on a tangent, or that the *extra* wear on a 6° curve is equal to that on a tangent, and also that the extra wear varies approximately as the *square* of the degree of curvature, which means that on a 10° curve the extra wear will be 300 per cent of the normal rail wear on a tangent. Even admitting the accuracy of this last statement, the rail wear is not proportional to the degree of curvature and therefore the total excess wear for each degree of central angle will not be uniform. But the value of 300 per cent for the excess rail wear on 528° in a mile will be used, and it will be shown later that an excessive variation from this estimate will not largely affect the final result.

Item 3: Ties work loose on curves more readily than on tangents, and, because they must be re-spiked more frequently, the ties are "spike-killed" before they decay. It has been estimated that a tie which will last nine years on a tangent will last six years on a 10° curve. If the life of other ties is in the same proportion, this will add 50 per cent for this item.

Among the items of maintenance of equipment items 11, 16, 18 and 19 may be at once struck out as being unaffected.

Item 12: Curvature affects engine repairs chiefly in the extra wear of wheels and in the straining and additional power required on curves. As the result of an elaborate analysis of engine repairs with an estimate of the proportion of them which are due to curvature, 6 per cent of all engine repairs are assigned to this cause. The average curvature on railroads amounts to about 30° per mile. If engine repairs varied directly as the amount of curvature, 528° per mile would increase the item for repairs by 17.6×6 or practically 100 per cent. Regardless of what the engine repairs might become on a 10° continuous curve, since we divide our final result by 528, we may consider that $\frac{1}{528}$ of the above figure will be correct for ordinary curves and therefore will add 100 per cent for the excess wear due to 528°. Although the effect of curvature on car repairs is likewise indefinite, the same value 100 per cent will be used for items 13, 14 and 15.

Item 17: The increase in this item will probably not exceed 50%.

Conducting transportation: Items 20, 21, 26, 27, 29 to 34, and 38 to 46 may at once be cut out.

Items 22 to 25: It was shown in §174 that about 50 to 55 per cent of these items are required to haul a train an additional mile and since the 528° of curvature is assumed to double the train resistance the increase in these items should be the same as that shown in Table XV.

Item 28: In general, curvature will not increase this item; in special cases extra flagmen are required to watch especially dangerous curves, but such cases may be allowed for if necessary;

no part of this item should be charged against curvature in general.

Items 35 to 37: As stated before in §177, a large part of these items have no relation to curvature; item 35 may be altogether ignored; it will probably be excessive to allow 50 per cent of items 36 and 37, but this will be done.

Items 47 to 53: These are evidently unaffected.

Collecting the above estimates and tabulating them we have

TABLE XVI.

Effect on Operating Expenses of Changes in Curvature.

Item No.	Normal average	Per cent affected	Cost per mile	Item No.	Normal average	Per cent affected	Cost per mile
[illegible]	10.779	25	2.69	20	1.759	0	0
[illegible]	1.442	300	4.33	21	9.640	0	0
[illegible]	[illegible]	50	1.53	22	9.949	55	5.47
				23	.649	50	.32
[illegible]	[illegible]	0	0	24	.372	50	.19
				25	.192	50	.10
	[illegible]		8.55	26 to 46	34.163	0	0
[illegible]	[illegible]	0	0				
[illegible]	15.712	100	15.71		56.724	—	6.08
[illegible]	[illegible]	0	0	47 to 53	4.428	0	0
[illegible]	[illegible]	50	.27				
[illegible]	[illegible]	0	0				
[illegible]	[illegible]	0	0				
	[illegible]		[illegible]		100.000	—	30.61

Considering the above percentages as *cents* per mile—since, as before, the average cost per mile is so nearly equal to $1—then we may say that 1° of central angle will increase the expense per train by [illegible] of 30.61 cents. Multiplying this by 2 × 365, we have 42 cents as the cost per year for each round trip of each daily train of 1° of central angle in a curve.

179. Numerical Example. *Reduction of curvature.* Assume that by a change of alignment, which is now common and familiar in the recent reconstruction of the great trunk lines, an easy alignment is substituted for a series of sharp reversed curves so that a sum total of 128° of central angle is reduced to 72°, thus saving 56° of central angle. Assume that there are sixteen daily trains each way. How much expenditure will be justified by this saving? On the above basis it will be 56 × 0.42 × 16 = $376.32. Capitalizing this at 5 per cent the justifiable expenditure to accomplish it is evidently $7,526. At 20 cents per cubic yard this would pay for excavating 37,600 cubic yards of earthwork. This is the equivalent of a cut 20 feet wide at the roadbed, slopes 1.5 : 1, over 12 feet deep, and 2000 feet long. Assume now that the estimate of rail wear is increased to 500 rather than 300 per cent as used. This would increase the estimate from 30.61 to 33.49, or an increase of 9.4 per cent. Such a change in the estimate is excessive and yet the effect on the final result is less than 10 per cent. If an engineer can rely on his economic calculations to within 10 per cent, he is immeasurably better off than he would be by a reliance on any "judgment" unsystematically formed.

180. Compensation for Curvature. Curvature makes a very definite increase in train resistance, and such increased resistance is readily equated to its equivalent in added grade. Assuming that the curve resistance on a 6° curve is 4 pounds per ton, which is the grade resistance on a 0.2 per cent grade, if there should be a 6° curve on a 1.0 per cent grade, the resistance on that grade would be the same as on a straight track having a 1.2 per cent grade. On this basis, if 1.2 per cent were selected as the ruling grade and it became necessary to introduce a 6° curve, the grade should be reduced on that curve to 1 per cent so that the total resistance on that curve shall be no greater than on the tangent. This is the fundamental idea of curve compensation. On grades which are so low that they will never be ruling grades even if the rate of ruling grade is reduced by reconstruction, there is no necessity for curve compensation, but the neglect of it on ruling grades means that the ruling grade is practically increased to the grade which is the equivalent of the combined grade and curve resistance, and that if the grade on the straight track is not increased to this virtual maximum, the advantages of a uniform ruling grade are lost. These advantages will be discussed later under "Grade."

Rate of compensation. This means such a reduction in the grade that the saving in grade resistance equals the curve resistance. But curve resistance varies somewhat as the velocity, the condition of the rails, and even the type of the wheel base. For simplicity of calculation the curve resistance is usually assumed to vary as the degree of curvature. While this is nearly true for low degrees of curvature, it becomes grossly inaccurate for excessively sharp curvature, on which the resistance is fortunately much less than its proportionate amount. This is probably due to the fact that a large part of the resistance from curvature is due to causes which are independent of the degree of curve. The resistance will amount to about 2 pounds per ton degree of curve (equivalent to a 0.1 per cent grade) when the velocity is very low—as when starting a train. It is less for fast trains than for slow trains, but considering that it is the slow and heavy freight trains which must be chiefly considered, the larger values for compensation which are needed for the slower velocities must be used. Compensation results in a loss of elevation for a given horizontal distance and when money has been spent in "development" in order to reduce the grade to some desired limit, any useless compensation is a waste and should be avoided. If a curve occurs on a grade immediately below a stopping place for *all* trains (or at least all trains which are so heavy that they will be affected by the ruling grade), the compensation may be reduced or omitted altogether on the ground that the curve resistance would simply use up the energy which might otherwise be used up by brakes in stopping the train. If that heavy grade should continue on above that stopping place, then the compensation should be made even greater than the average to allow for the increased resistance while starting. Since the curve resistance merely adds to the virtual grade, and the object of compensation is to prevent such additions from increasing the ruling grade, there is no object in using compensation on a grade, which is already so low that the added resistance will not make it virtually equal to the ruling grade. An exception to this lies in the danger that it may some time prove desirable to make such changes of alignment that the ruling grade is very materially cut down, and it might happen that neglect to compensate would add that much to the revised ruling grade. The above discussion may therefore be reduced to the following rules:—

(1) On the upper side of a stopping place for all heavy trains compensate 0.10 per cent per degree of curve.

(2) On the lower side of such a stopping place do not compensate at all — but this rule should be applied cautiously.

(3) Ordinarily compensate about 0.05 per cent per degree of curve.

(4) Reduce this rate to 0.04 per cent and possibly to 0.03

per cent if it is found that the higher rate of compensation causes such a loss of height that the grade on the tangent must be increased.

(5) Reduce the rate somewhat for curvature above 10°.

(6) Curves which are so much less than the ruling grade that they will always be minor grades need not be compensated.

181. Limitations of Curvature. Surveys for railroads are frequently made under instructions that curves (and also grades) shall not exceed some chosen limitations. What should be the limitation, if any, of the degree of curvature? Probably no definite answer is correct unless it be said that there should be no limitation. It has been shown that all ordinary degrees of curvature even up to 20° will still permit the use of heavy engines, and there are numerous instances where a heavy railroad traffic has been hauled for many years around excessively sharp curves without any serious difficulty—as, for instance, the traffic on the Baltimore & Ohio R. R. at Harper's Ferry which for many years was hauled around a 19° 10′ curve (radius 300 feet). This curve has recently been changed. Of course the young engineer should not conclude from this that curvature is of no consequence, and that he may recklessly put in as much and as sharp curvature as might seem at first the easiest plan to adopt. The preceding discussion shows that there is a definite money value in reducing every possible degree of central angle and also that the radius of curvature should be made as large as possible without a serious sacrifice of other interests or extravagant expenditure. It generally happens, when running a road through a mountainous country, and when a high summit must be crossed, that the grades can only be reduced by the adoption of very sharp curvature or by a large expenditure in construction. Since the expenditure is usually limited by financial considerations the error of adopting a high ruling grade is usually made and the degree of curvature is limited to a low figure which is ridiculously out of proportion with the general condition of the road.

Sometimes the limited money at the disposal of the company is wasted on a route which gives easy curves when the money could have been spent advantageously in other ways. But the more common error is the needless increase in the ruling grade. Many railroads have been laid out under the instructions that the maximum grade may be 60 feet per mile and the maximum curve 6°. These limits have been used separately or in combination, with the result that when a 6° curve occurred on a 60-foot grade, the virtual grade was thereby increased (on a 0.05 per cent basis) to nearly 76 feet per mile. While a grade of 60 feet per mile might be a very proper ruling grade under certain conditions, it might readily happen that the option of using a 10° or even 20° curve would permit adopting a line with a ruling grade so much less than 60 feet that the advantages of the reduction of grade would far outweigh the comparatively insignificant disadvantages of the sharp curvature. Therefore, as a general answer it may be said that the limits, if any, should conform to the general character of the country, and that when it appears possible to obtain a great advantage, such as the reduction of the ruling grade, by an increase in the degree of curvature and even in the degrees of central angle, such increase should be made unless it may be definitely computed that the disadvantages of the increased curvature would outweigh the advantages of the reduced grade.

GRADE.

182. Distinction between Minor and Ruling Grades. This distinction must be very clearly understood before their operating disadvantages may be computed. The previous discussion has shown that the cost of running a train one mile is approximately a constant quantity, no matter whether the train is long or short, heavy or light. The receipts for transporting so many tons of freight is a definite quantity and is unaffected whether it is transported in one train load or two. If it is possible by a reduction in grade to haul in a single train load as much freight as would require two train loads by the old plan, then, since the receipts are constant and the cost of the two light trains will be nearly double that of the one heavy train, it is evident that the low-grade plan will be very profitable and the other plan correspondingly costly and financially ruinous.

Although it is not often practicable to double the weight of the train behind a freight engine, a very material increase in the train load can generally be made by such reduction of the ruling grade as is practicable, and such increase in train load frequently makes all the difference between large dividends and an actual deficit. The ruling grade definitely limits the load that can be hauled by an engine with a given weight on the drivers and its financial effect is very great. On the other hand, a minor grade does not limit the number of cars and its effect on operating expenses is confined chiefly to an increase in the consumption of fuel and other locomotive supplies. While this increase in expense has an importance which is worth computing, it is insignificant compared with the cost of running additional trains to handle a given traffic.

The real cost of minor grades is also less than it might otherwise be considered owing to the fact that each rise has its corresponding fall. Even though several high summits may be crossed, the difference of elevation of the termini (say 200 or even 500 feet) is insignificant from the standpoint of grade when the distance is perhaps as many miles. And even in the extreme case when the grade is all in one direction, the additional energy required to climb the grades is partly returned in the assistance the grade gives to trains on the return trip and the consequent saving in motive power.

183. Laws of Accelerated Motion. *Application to movement of trains.* When a train starts from rest and acquires its normal velocity (say 30 miles per hour) the engine must develop not only the power required for all the ordinary tangent and perhaps curve and grade resistances, but also the "kinetic energy" corresponding to the velocity which has been acquired. This kinetic energy is not wasted; all of it is transformed back into work of some kind. The work may be consumed and wasted in the brakes, but it may also be spent (and is so spent) in overcoming resistances whenever the velocity of the train is reduced. The amount of this kinetic energy is a definite mathematical quantity. The laws of Mechanics tell us that this energy equals $W \frac{v^2}{2g}$, in which $W =$ the weight of the train, $v =$ its velocity in feet per second and $g =$ the acceleration of the force of gravity, which equals 32.16 feet per second in a second.

A better appreciation of this force may be obtained by considering for a moment that if the train could move along the track without any resistance, then, when running at a velocity of v feet

per second, it possesses a kinetic energy which would raise it to a height of h feet, where $h = \frac{v^2}{2g}$. If we consider that the engine is furnishing exactly the power required to overcome the tractive resistances, then the train would run until it had climbed a grade to a height of h feet, no matter whether it was accomplished in 100 feet or a mile. By an expansion of the theory it is also shown that when the train has climbed a vertical height of h' feet (less than h), it will have a velocity $v' = \sqrt{2g\,(h-h')}$.

For example, assume that the velocity of the train is 30 miles per hour, = 44 feet per second. It then has a kinetic energy which would raise it a height in feet $= h = \frac{44^2}{2 \times 32.16} = 30.1$ feet. If the engine furnished just enough energy to overcome the tractive resistances, the kinetic energy would carry the train up a grade of 15 feet per mile for a distance of about two miles or up a grade of 60 feet per mile for a distance of about a half mile. If the train were moving up a grade of 20 feet per mile and had proceeded half a mile, it would have climbed ten feet and would still have a kinetic energy corresponding to 20.1 feet, and its velocity would then $= v' = \sqrt{2 \times 32.16 \times 20.1} = 35.9$ feet per second.

If the train were a solid mass the above figures would be absolutely correct, but the solution is a little complicated by the fact that an appreciable part of the weight of the train consists of revolving wheels, to which must be imparted the kinetic energy of rotation in addition to the kinetic energy of translation. The ratio of this rotative kinetic energy to that of translation depends chiefly on the ratio of the weights of the wheels and of the whole car or engine. Evidently this ratio depends on the detailed design of the rolling stock, and more especially on whether the cars are loaded or empty. This consideration shows that no one value will be accurate for all cases, but there will be little error in adopting 5 per cent as an average value for the increase in the kinetic energy.

Table XVII, which will be found very useful in these computations, has therefore been computed on the following basis:

$$\text{"Velocity head"} = \frac{\text{V}^2 \text{ in ft. per sec.}}{2 \times 32.16} = \frac{1.4667 \text{ V}^2 \text{ in m. per h}\cdot}{64.32}$$

$$= 0.03344\text{V}^2$$

adding 5 per cent for the rotative kinetic energy of the wheels, $= 0.00167\text{V}^2$

The corrected velocity head therefore $= 0.03511\text{V}^2$

TABLE XVII.

Velocity Head (Representing the Kinetic energy) of Trains Moving at Various Velocities.

Velocity. miles per hours.	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
10......	3.51	3.58	3.65	3.72	3.79	3.87	3.95	4.02	4.10	4.17
11......	4.25	4.33	4.41	4.49	4.57	4.65	4.73	4.81	4.89	4.97
12......	5.06	5.15	5.23	5.32	5.41	5.50	5.58	5.67	5.75	5.84
13......	5.93	6.02	6.12	6.21	6.31	6.40	6.50	6.59	6.69	6.78
14......	6.88	6.98	7.08	7.19	7.29	7.39	7.49	7.60	7.70	7.80
15......	7.90	8.00	8.11	8.22	8.33	8.44	8.55	8.66	8.77	8.88
16......	8.99	9.10	9.21	9.32	9.43	9.55	9.67	9.71	9.91	10.03
17......	10.15	10.27	10.39	10.51	10.63	10.75	10.87	10.99	11.12	11.25
18......	11.38	11.50	11.63	11.76	11.89	12.02	12.15	12.28	12.41	12.55
19......	12.58	12.81	12.95	13.08	13.22	13.35	13.49	13.63	13.77	13.91
20......	14.05	14.19	14.33	14.47	14.61	14.75	14.89	15.04	15.19	15.34
21......	15.49	15.64	15.79	15.94	16.09	16.24	16.39	16.54	16.69	16.84
22......	17.00	17.15	17.30	17.46	17.62	17.78	17.94	18.10	18.26	18.42
23......	18.58	18.74	18.90	19.06	19.22	19.38	19.55	19.72	19.89	10.06
24......	20.23	20.40	20.57	20.74	20.91	21.08	21.25	21.42	21.59	21.77
25......	21.95	22.12	22.30	22.48	22.66	22.84	23.02	23.20	23.38	23.56
26......	23.74	23.92	24.10	24.28	21.46	24.65	24.84	25.03	25.22	25.41
27......	25.60	25.79	25.98	26.17	26.36	26.55	26.74	26.93	27.13	27.33
28......	27.53	27.73	27.93	28.13	28.33	28.53	28.73	28.93	29.13	29.33
29......	29.53	29.73	29.93	30.13	30.34	30.55	30.76	30.97	31.18	31.39
30......	31.60	31.81	32.02	32.23	32.44	32.65	32.86	33.08	33.30	33.52
31......	33.74	33.96	34.18	34.40	34.62	34.84	35.06	35.28	35.50	35.72
32......	35.95	36.17	36.39	36.62	36.85	37.08	37.31	37.54	37.77	38.00
33......	38.23	38.46	38.69	38.92	39.15	39.38	39.62	39 86	40.10	40.34
34......	40.58	40.82	41.06	41.30	41.54	41.78	42.02	42.26	42.51	42.76
35......	43.01	43.26	43.51	43.76	44.01	44 26	44.51	44.76	45.01	45.26
36......	45.51	45.76	46.01	46.26	46.52	46.78	47.04	47.30	47.56	47.82
37......	48.08	48.34	48.60	48.86	49.12	49.38	49.64	49.91	50 18	50.45
38......	50.72	50.99	51.26	51 53	51.80	52.07	52.34	52.61	52.88	53.15
39......	53.42	53.69	53.96	54.23	54.51	54.79	55.07	55.35	55.63	55.91
40......	56.19	56.47	56.75	57.03	57.31	57.59	57.87	58.16	58.45	58.74
41......	59.03	59.32	59.61	59.90	60.19	60.48	60.77	61.06	61.35	61.62
42......	61.94	62.23	62.52	62.82	63.12	63.42	63-72	64.02	64.32	64.62
43......	64.92	65.22	65.52	65.82	66.12	66.43	66.74	67.05	67.36	67.67
44......	67.98	68.29	68.60	68.91	69.22	69.53	69.84	70.15	70.46	70.78
45......	71.10	71 42	71.74	72.06	72.38	72.70	73.02	73.34	73.66	73.98
46......	74.30	74.62	74.94	75.26	75.59	75.92	76 25	76.58	76.91	77.24
47......	77.57	77.90	78 23	78.56	78.89	79.22	79.55	79.89	80.23	80.57
48......	80.91	81.25	81.59	81.93	82.27	82.61	82.95	83.29	83.63	83.97
49......	84.32	84.66	85.00	85.34	85.69	86.04	86.39	86.74	87.09	87.44
50......	87.79	88.14	88.49	88.85	89.20	89.55	89.91	90.26	90.61	90.97

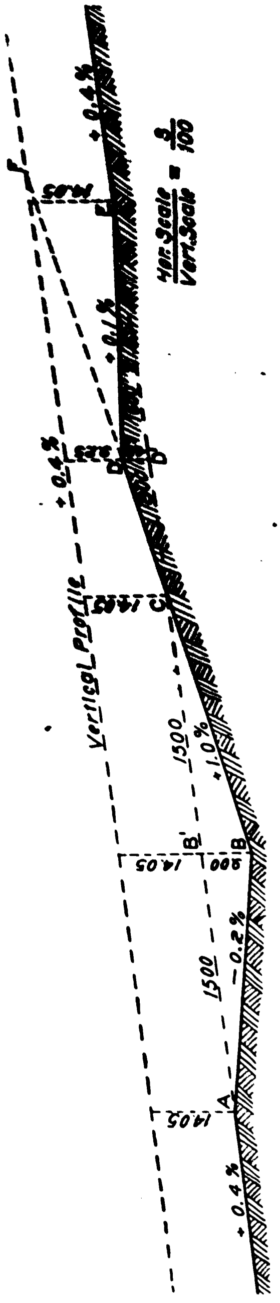


Fig. 158.

Part of the figures of Table XVII were obtained by interpolation, and, therefore, there *may* be an error of a single unit in the hundredths place in some of the figures, but considering the uncertainties in the problem, the exact value to hundredths is of no practical importance. Examples of the application of this table will be given later.

184. Virtual Profile. **The following demonstrations are made on the basis that the ordinary tractive resistances and also the tractive force of the locomotive are independent of velocity. Although this is not strictly true, it is so nearly true that calculations made on this basis are applicable to freight train problems in which the velocity is usually low. Trains which must make fast time must have a very great excess** of power so that they may accumulate velocity quickly and, therefore, the necessity for accuracy is not so great. Assume that Fig. 158 shows the profile of a section of road and that the grade of A E is 0.40 per cent, which is 21.12 feet per mile. Assume also that a freight engine which is climbing up the grade has been so loaded that when the engine is working *uniformly* at its normal maximum the velocity up such a grade would be uniformly 20 miles per hour. But since the train is moving at 20 miles per hour it has a kinetic energy corresponding to a velocity head of 14.05 feet (see Table XVII). At A it encounters a down grade of 0.20 per cent, which is 1500 feet long. Although

A B has a down grade of only 0.2 per cent, its grade with respect to the up grade of A E (0.40 per cent) is 0.60 per cent. Therefore B is 9.00 feet below B'. Since the work done by the engine would have carried the train up to the point B' with a velocity of 20 miles per hour, the *virtual* drop of 9 feet will increase the velocity head from 14.05 feet to 23.05 feet, which corresponds to the velocity of 25.6 miles per hour, and this will actually be the velocity of the train at the point B. At B the grade changes to a 1.0 per cent up grade for a distance of 2300 feet.

The approach of the grade B C to the grade B' C is at the rate of (1.0–0.4) or 0.6 per cent and therefore the point C will be reached in 1500 feet. In the remaining 800 feet the line will climb to D, which is 4.8 feet above D'. Although at B the train is moving at the rate of 25.6 miles per hour and the engine is working at such a rate that it will carry the train up a 0.4 per cent grade, yet when climbing up a 1.0 per cent grade it consumes its kinetic energy in overcoming the additional grade. When it reaches C, it has lost the additional kinetic energy which it gained from A to B, and as it continues it loses even more. When it reaches D it has lost 4.8 feet more and its velocity head is reduced to (14.05–4.8) = 9.25 feet, which corresponds to a velocity of 16.2 miles per hour. At D the grade changes to + 0.1 per cent.

Here we have the rather surprising condition that, although the grade is actually rising, it is virtually a down grade under the given conditions, for the engine is working harder than is required to run up merely a 0.1 per cent grade and hence will gain in velocity. At E, a distance of 1600 feet from D, it reaches what would have been a uniform 0.4 per cent grade from A to E and the grade continues at that rate. Although the train has actually climbed 1.6 feet from D to E, it has virtually fallen the 4.8 feet between D and D', and the velocity head has increased from its value of 9.25 feet at D to 14.05 feet, and its velocity is again 20 miles per hour. The upper line represents the "virtual profile," which may always be drawn by measuring off to the proper scale at every point an ordinate which is the velocity head at that point. Since the engine is working uniformly, the virtual profile is in this case a straight line.

Although the variation of resistance and tractive effort with velocity will have some effect on the precision of the above figures, yet it must not be considered as fanciful and impractical. Under the given conditions it is substantially what would take place. If the grade B D were continued to F, or until the actual grade intersected the virtual profile, the train would become stalled, for when the engine is loaded for an indefinite haul up a 0.4 per cent grade, it cannot haul a train indefinitely up a higher grade. Practically the train would stall somewhat short of F, since the tractive resistance increases as the velocity drops to nearly zero. Under such conditions, B D is a "momentum grade," which *may* be higher than the ruling grade, and yet it is practically harmless under these conditions, *provided* that a train is never required to stop on that grade. A B C is technically a "sag" in the grade A C and would be considered such even if A B were an up-grade (although less than the grade A C). Such a sag is usually harmless unless it is so deep that the train would acquire a dangerously high velocity at the bottom of the sag B.

In the above numerical case the velocity is only 25.6 miles per hour, which is not at all dangerous even for freight trains in these days of air brakes and automatic couplers. But a much deeper sag might require the use of brakes, which not only consumes some of the energy stored in the train, but also wears out the wheel treads and brake shoes.

Another phase of the question is developed when we consider the action of a train stopping on a grade. Assume, as in Fig. 159, that a train is climbing up the grade A B at a uniform velocity whose velocity head is measured by A A′ = B B′. At B it commences to slow up for a stop at C. Since it is stationary at C, the velocity head is zero and the virtual profile A′ B′ runs from B′ to C by a line which may or may not be straight. Assume that the train starts up and the engine exerts such force that at D it has regained the velocity it had at A or B. The ordinate DD′ must equal A A′ and the virtual profile must run from C to D′. C D′ therefore represents the virtual grade up which the train must climb. To put it in figures; assume that C D = 1,300 feet; the required velocity at D is 20 miles per hour, and therefore D D′ = 14.05; the grade of C D is 1.0 per cent and therefore D D″ = 13 feet, and D'' D' = 27.05 feet; the virtual grade C D' is therefore 2.08 per cent instead of the actual 1.0 per cent and these figures represent the actual ratio of the drawbar pulls at the engine.

To be more precise, the virtual grade C D' will not be a uniform grade as shown in the figure but will be a curved line which will be steeper at the beginning of the grade on account of the increased resistance to traction when starting. This is somewhat compensated by the fact that the tractive force of the engine is greater at the very low velocities It requires, however, a little margin for safety.

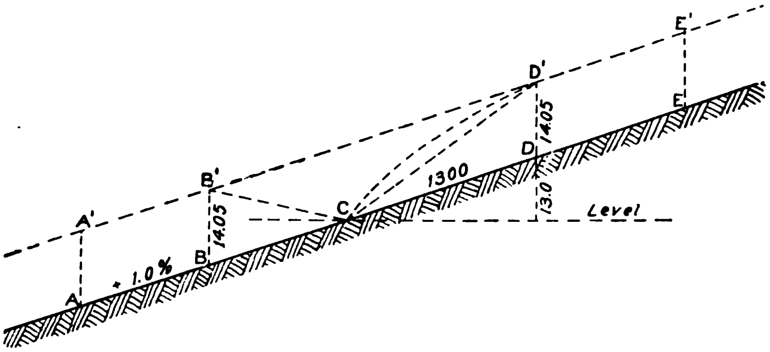


Fig. 159.

The fact that the engine can increase its velocity from zero to 20 miles per hour in that distance and on that grade shows that it is capable of doing much more than run its train up the 1.0 per cent grade at a speed of 20 miles per hour. In fact, unless the power is reduced when the train reaches D, the train will continue to gain velocity. If resistance were independent of velocity, the train would continue to gain indefinitely assuming that the grade continued uniform. But practically, when the velocity had increased to a much higher figure, the resistances would increase until even the added power could not further increase the velocity.

From all the above it may be inferred that

(*a*) When the velocity is uniform, the virtual profile is parallel with the actual profile.

(*b*) When the velocity is increasing, the profiles are separating; when it is decreasing they are approaching each other.

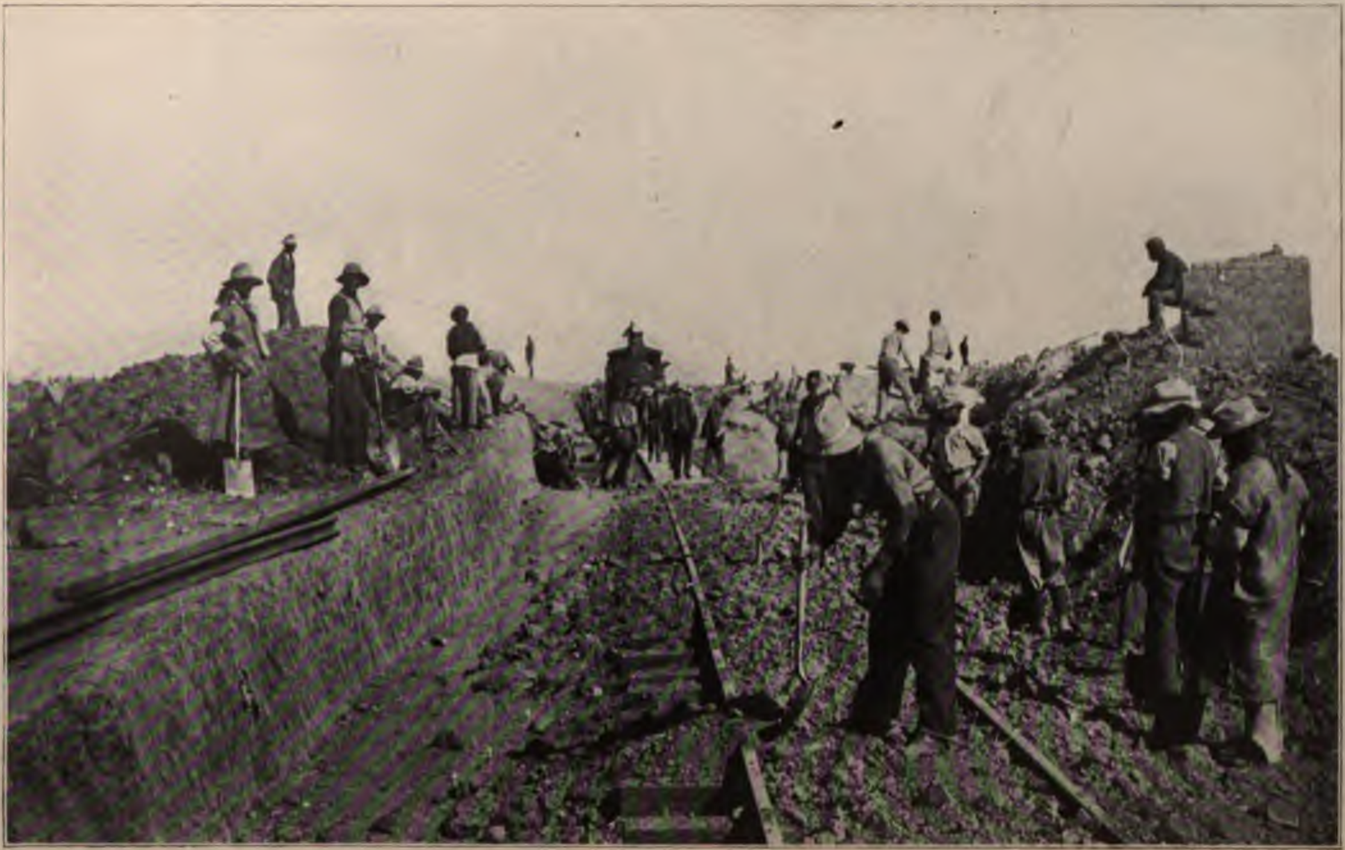
(*c*) When the velocity is zero the profiles coincide.

(*d*) The virtual grade at any place is a measure of the work required of the engine beyond that required to overcome merely the tractive resistances. If it is horizontal it shows that the engine is doing nothing beside overcoming the tractive resistances. If it is upward and is uniform, as in Fig. 158, it shows that it is working uniformly and is storing in the train "potential" energy which may be utilized on the return trip if it is not utilized in moving down a succeeding down grade. If it is downward (as from B′ to C, Fig. 159) it shows that the train is giving up kinetic energy, probably consuming the most of it in brakes, but utilizing some of it to furnish the tractive power to run from B to C and also to overcome the grade from B to C.

185. Use, Value, and Possible Misuse of Virtual Profiles. It has been previously shown that, aside from securing the maximum traffic, the most important accomplishment for the locating engineer is to obtain low ruling grades. At the same time the cost for grading must be kept as low as possible without sacrificing the more important elements. The grade B D in Fig. 158 is an example of the possibility of introducing a grade which is much steeper than the ruling grade, provided that it is not so long that the kinetic energy in the train at the bottom of the grade shall not be exhausted before it reaches an easier grade, and also provided that no heavy trains shall ever be compelled to stop on that grade. Herein lies the danger and the possible misuse of this method.

A grade might be laid out substantially as shown in Fig. 158, with the intention of running all heavy trains up that grade without stopping. Later another railroad might require and make a grade crossing at or near C, which would occasionally require that trains shall stop at the crossing, and such loaded trains would be unable to start against such a grade, especially since the tractive resistance to starting is so much greater than the resistance at ordinary speeds. The chief value of such a method lies in the fact that it enables the engineer to determine the actual demand on the locomotive, as it is affected by the velocity of the train. The "undulatory" profile shown in Fig. 158 will probably be much cheaper to construct than the uniform grade A E which would involve a fill at B and a cut at D. The method of a virtual profile will show at once whether such a profile at that place will be a

PUR



RAILROAD BUILDING IN ECUADOR

permissible way of economizing in spite of the fact that it introduces a 1 per cent grade which is perhaps higher than the ruling grade. Many of the "improvements of old lines" which are described later depend on this process for their solution.

For example, a grade which may have always been harmless and unnoticed suddenly becomes important when it becomes desirable or necessary to require all heavy trains to stop at some point on it; such a case as is sketched in Fig. 159. The above method indicates how such a problem may be investigated. The grade C D′ of course becomes the critical grade, but under given conditions the virtual profile will show the demand on the locomotive. Examples of this will be given later. Undulatory grades have the advantage of decreasing the cost of construction and of being harmless under given conditions, but there are some dangers. C D E in Fig. 158 is called a "hump" in the grade. In the numerical case given it is only 4.8 feet and would be harmless under almost any conditions, but if it were considerably more, and if a train when passing C had a velocity much less than 20 miles per hour, it might become stalled before reaching the summit of the hump.

Slippery rails or a strong head wind may so increase the resistances against which a train works that if the computed margin of velocity head at the top of a hump is made too small it may be entirely overcome and the train may be stalled before it is safely over the hump. A velocity of 10 miles per hour, which corresponds to a velocity head of only 3.51 feet is the least margin that should be safely allowed. This is also partly due to the fact that when the velocity becomes much less than ten miles per hour the resistances per ton increase, and as the velocity drops very low they increase very rapidly and the law on which the above calculations are based becomes inoperative. Another danger is that a sag may be so deep that trains will acquire an excessive velocity when passing through it unless brakes are applied. This of course does not mean that the sag must not be used. It simply means that the sag will cause a waste of energy in brakes, a waste which must afterward be made up by increased work from the locomotive. This is of course one of the cases which requires computation, by methods which follow, to determine whether or to what extent the sag is justifiable so that the two items of increased first cost and increased operating expenses shall be made a minimum.

For example, a freight train may approach a sag with a velocity of 20 miles per hour. Its velocity head is therefore 14.05 feet. If the sag at its lowest point is 40 feet lower than the imaginary grade line on which the train could have run without changing its velocity (the grade A B′ in Fig. 158) then the velocity head of the train at the bottom of the sag would be 54.05 feet which corresponds to a speed of 39.2 miles per hour. Although this is a permissible speed with freight trains which are equipped with air brakes and automatic couplers, it is approaching the limit, and there might be some local conditions which would render even this speed through the sag inadvisable.

186. Problems. 1. If a train is running uniformly along a level grade at a speed of 35 miles per hour and reaches a 1.2 per cent up grade, how far up the grade could it run before its speed is reduced to 10 miles per hour?

Velocity head for 35 miles per hour = 43.01 feet
Velocity head for 10 miles per hour = 3.51 feet

Permissible increase in elevation = 39.50 feet
Distance from bottom of grade = 39.50 ÷ .012 = 3,292 feet.

2. At what speed may a train approach a sag 28 feet below the normal grade line so that its maximum speed at the bottom of the sag shall not exceed 36 miles per hour?

At 36 miles per hour the velocity head = 45.51 feet
Subtracting the depth of the sag = 28.00 feet

The permissible velocity is that due to 17.51 feet and = 22.3 m. per h.

MINOR GRADES.

187. Basis of Cost. As in the method of computing the additional cost of curvature, we compute the rate of grade which will double the tractive resistance, and then compute the effect of this on all the items of the cost of a train mile. While the resistance to grade is a perfectly precise mathematical quantity, ordinary tractive resistance on a level track depends on the velocity, the number and character of the cars, the physical condition of their wheel treads, and the condition of the rails, as well as many other minor causes. But as an average figure we may say that for ordinary train velocities the train resistance may be considered as 10 pounds per ton. But this is the grade resistance on a 0.5 per cent grade, or a grade of 26.4 feet per mile. On this basis the total resistance on a 0.5 per cent grade will be just double the resistance on a level. We may compute as before the effect on operating expenses of doubling the work done by the locomotive. But this added cost will be the cost of lifting the train 26.4 feet in one mile. If we divide the extra cost of a mile of 0.5 per cent grade by 26.4 we will have the added cost of *one foot* of elevation. Then, since it is assumed that the grade has no limiting effect on trains, the cost of one foot of difference of elevation will be independent of the rate of grade, except as noted later.

188. Classification of Minor Grades. Even the minor grades have varying effects on the operation of trains which are sharply defined and which have varying effects on the operating expenses. Under the conditions named in the problem in §184, Fig. 158, the variations in grade are harmless since it is assumed that they do not cause any variation in the drawbar pull. But it will frequently happen that a hump or sag that will be harmless for a light fast passenger train will have a very serious effect on a heavy freight train which always runs at slow speed and which therefore never has an opportunity to accumulate any great excess of kinetic energy which may assist it over a hump of any magnitude. We will therefore consider as Class A all fluctuations of grade which are so slight that they have no effect on the running of trains except a slight variation of the velocity. One of the practical limitations of Class A is the maximum depth of a sag rather than the actual grade. If the depth of the sag is so great that even with a slow approach to it, the velocity at the bottom will be too great for safety, and steam must be shut off and brakes must be applied, then the sag cannot belong to Class A.

Class B is an intermediate class in which the loss due to grade is not so severe. A down grade of 0.5 per cent will keep a train running indefinitely without steam power. If steam is used on such a grade the speed would increase until it would become excessive if the grade were long Therefore in running down such a grade, steam would generally be shut off. One disadvantage of this lies in the fact that some fuel will be consumed even then, and it is more economical to have the locomotive working uniformly, provided its work may be stored up as kinetic energy which can be utilized in climbing the succeeding up grade. Such a grade also means that the tractive effort when returning up that grade will be at least twice the required pull on a level. The loss in this case is not so serious; there is no actual waste of energy in brakes, and the loss consists chiefly in an inability to utilize the power of the locomotive to the best advantage.

Class C includes grades which are so steep or so long that brakes must be applied in descending. This also means that the power required for ascending will be excessive. In this case there is not only a waste of energy which has been developed by the locomotive, but there is also the added expense of wear of brake shoes and of wheel tires (due to the action of the brakes), and even some extra wear and tear on the rolling stock due to the extra stresses, and also the extra wear of rails.

The only possible practical application of the following computations lies in the comparison of two proposed routes between points at a greater or less distance apart. The simplest case is that of a mere hump or sag in comparison with a uniform grade between the given points. If the distance between the points is very great so that the character of the alignment of the two routes is very different, although any differences in distances or curvature should in any case be computed separately, it may be difficult to properly classify the effect of the differences of grade of the two lines. The following cases will only consider the simple cases of sags and humps. In each case there is involved an up grade and a down grade (virtually if not actually) and we must compute the effect of both on the passage of any one train and then this is multiplied by two to compute the effect for each daily round trip of a train.

189. Effect on Operating Expenses of a Hump or a Sag. As before, such a very large proportion of the items of the cost of a train mile are unaffected that the simplest plan will be to throw out first the unaffected items. Of the items of maintenance of way items 4 to 10 can be at once thrown out.

Item 1: A very large proportion of the sub-items are evidently unaffected; the wear of ties and rails is somewhat affected when the grade is excessive, but even this is somewhat compensated by the improved drainage of the track. Therefore, no allowance is made for Class B and only 5 per cent is allowed for Class C.

Item 2: Rail wear on grades is noticeably greater than on a level. This is partly due to the use of sand, but is also due to some other cause which is not very clear, since the rails on the down grade of a double-track road are found to be badly worn. The skidding of wheels, if such were permitted, would account for it, but this is usually carefully guarded against. But since it is an undoubted fact allowance must be made for it. An allowance of 5 per cent is made for Class B and 10 per cent for Class C.

Item 3: Considering the compensation of better drainage on a track having a slight grade, no increase in the cost of tie renewals is made for Class B and only 5 per cent is allowed for Class C.

Maintenance of equipment. All items are at once cut out except those of actual repairs to rolling stock. Even here a very large proportion of the sub-items should evidently be cut out. The increase will be chiefly on account of an increase in wheel and brake-shoe wear, with a possible addition for drawbar gear, and also some addition for locomotive repairs due to additional work on the grades. A very large proportion of even these sub-items is due to the effects of stopping and starting, curvature, straight level pulling, etc., and, therefore, it is not surprising that Wellington, after an exhaustive investigation, cut down these items to 1 per cent for Class B and 4 per cent for Class C.

Conducting transportation. All items except the various locomotive supplies are immediately cut out. It was shown in §174 that about 55 per cent of the fuel consumption was due to straight hauling on a tangent. But since the grade here considered is assumed to be that which will exactly double the resistance on a straight level tangent, we will add 55 per cent for the grade. But we must here compute the cost of both the rise and the fall. In Class B, although the steam is shut off, heat is wasted by mere radiation, which has been shown to be responsible for about 5 per cent of the total consumption; for Class C we must allow 5 per cent extra for the extra energy spent in applying brakes; this makes

60 per cent for Class B and 65 per cent for Class C. There is but little error in assuming that items 23, 24 and 25 will be affected similarly. *General expenses* will be unaffected.

Table XVIII in which these items are collected shows that the percentage of increase for operating 26.4 feet of rise and fall will be 6.92 and 8.71 for Classes B and C respectively. As before we may call these *cents* and then say that the cost per foot will be 0.262 cents and 0.330 cents. For each daily train each way the cost per year will be

For Class B: 2 × 365 × $0.00262 = $1.91
For Class C: 2 × 365 × $0.00330 = $2.41

TABLE XVIII.

Effect on Operating Expenses of 26.4 Feet of Rise and Fall.

No.	Item (abbreviated) *	Normal average	Class B.		Class C.	
			Per cent affected	Cost per mile	Per cent affected	Cost per mile
1	Roadway	10.779	0	0	5	.54
2	Rails	1.442	5	.07	10	.14
3	Ties	3.060	0	0	5	.15
4-10	Bridges, buildings ...	5.924	0	0	0	0
	Maintenance of way ..	21.205	—	.07	—	.83
11	Superintendence	.631	0	0	0	0
12	Repairs, locomotives	6.258	1	.06	4	.25
13	" , pass. cars ..	2.218	1	.02	4	.09
14	" , freight cars.	7.048	1	.07	4	.28
15	" , work cars...	.188	1	0	4	.01
16-19	Miscellaneous	1.300	0	0	0	0
	Maintenance of equip.	17.643	—	.15	—	.63
20	Superintendence	1.759	0	0	0	0
21	Enginemen	9.640	0	0	0	0
22	Fuel	9.949	60	5.97	65	6.47
23	Water	.649	60	.39	65	.42
24	Oil, etc	.372	60	.22	65	.24
25	Other supplies	.192	60	.12	65	.12
26-46	Train and station service, switchmen, etc.	34.163	0	0	0	0
	Conducting transport	56.724	—	6.70	—	7.25
47-53	General expenses	4.428	0	0	0	0
		100.000	—	6.92	—	8.71

* For full title of item see Table XIV.

190. Numerical Example. Assume the data as sketched in Fig. 160. If a freight train moving to the right passes the point B at a velocity of 15 miles per hour, it will have a velocity head of 7.90 feet. Even if the engine were only doing sufficient work to keep it moving on a level track, it will have gained in velocity head the full amount of the vertical drop from B to C by the time it reaches C. Its velocity head will then be 84.7 feet which corresponds to a velocity of over 49 miles per hour which is plainly inadmissible. Assume that steam is shut off at B. Then, on the basis that tractive force requires a grade of 0.5 per cent, we may consider that the 1.2 per cent grade is a 0.7 per cent down grade which the train passes without any tractive resistance. On this basis the virtual fall from B to C is 32.0 feet, and the velocity head at the point C would be 39.90, which corresponds to a velocity of 33.7 miles per hour. This may or may not be a permissible velocity, but considering that even this grade is followed by a 0.7 per cent grade on which the velocity would increase still further, it is evident that brakes would be necessary on some trains if not on all.

The possible limiting effect on the length of trains moving to the left of the grade C B is a distinct matter and must be here ignored. But it is evident that unless a train is moving at a speed of over 30 miles per hour or else is working much harder than is required for a 0.7 per cent grade, it would not have sufficient velocity head to carry it over the summit at B. On the above basis per foot the cost per daily train each way per year will be

32.0 × \$1.91 = \$61.12
for all trains of Class B, and
32.0 × \$2.41 = \$77.12
for all trains of Class C.

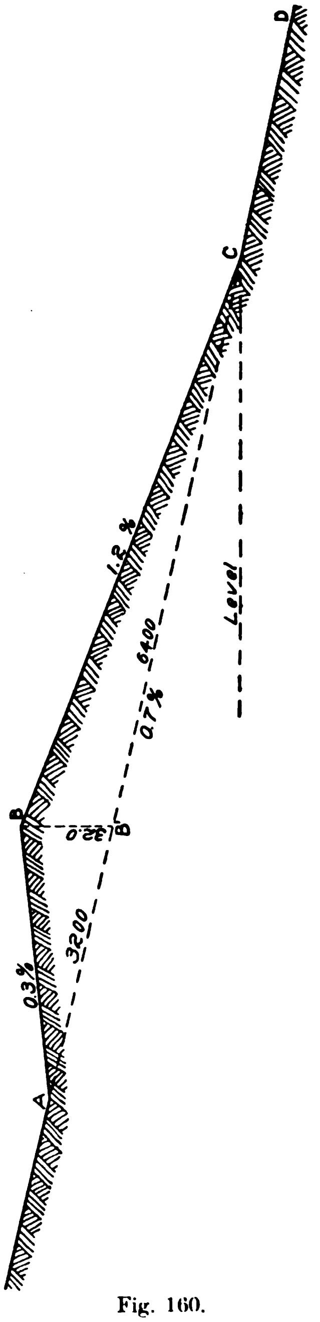


Fig. 160.

Classifying all the trains on the road, we may say that there are some light passenger trains which may be permitted to run at such speed that this grade may be considered harmless and therefore for them it may be considered as belonging to Class A. Assume that there are four other trains for which it must be considered as belonging to Class B and six others for which we must consider it as Class C. Then we have

$61.12 × 4 = $244.48
77.12 × 6 = 462.72

Total annual value = $707.20

This represents a capitalized value, at 5 per cent, of $14,144. Considering for the sake of an approximate value that the sections were level and that a cut was to be made 20 feet wide at the base and with slopes of 1.5 : 1, the cutting necessary to reduce the grade to the line A B′ C would be about 296,000 cubic yards. Unquestionably $14,000 will not pay for this excavation, but when it is considered that the quantity of earthwork required for any depth of cutting increases almost as the square of the depth and that a cut of half the depth may be made by the excavation of perhaps only one-third of the earthwork, it is possible that enough could be cut from the summit B to change the classification of the grades, that the hump could be made so slight that it would not be worse than Class B for any train and would be Class A (or harmless) for nearly all the trains. An advantage of this method of work lies in the facility for selecting various plans and computing the effect of each on the operation of trains.

The chief difficulty with the above method of computation lies in the classification of the grades, or in the assumption of the extent of the improvement accomplished by the change. It is conceivable that a hump or sag might be removed and yet the grade as left would have all the characteristics of Class C. But even in such a case, an improvement has certainly been made; a very bad condition has been improved even though it is still serious. It is therefore justifiable to consider that the value of the improvement may be measured as above even if the basis is somewhat changed. As before, it is not a question of close accuracy, but of developing a basis of computations which approaches the truth and which may be made still more accurate by a closer study of the conditions.

RULING GRADES.

191. Definition. Ruling grades are those which limit the length or weight of trains which may be hauled by a single engine. They are not necessarily the steepest nominal grades on the road, for we have already seen that a momentum grade may be much steeper and yet not affect the admissible weight of a train. Another exception is a "pusher grade," on which it is designed that all heavy trains shall be assisted by extra engines. This condition requires separate calculation which will be given later; it involves extra expense, but has no greater effect on limiting trains than the corresponding grade for one engine, and of course all pusher grades are so designed that the corresponding grade for a single engine shall not exceed the ruling grade. In general, an engineer knows that the ruling grade must be made as low as the general financial character of the enterprise and the general character of the country will permit.

By their very nature, the following calculations are only applicable to the improvement of lines already built or to the change of plans which have already been made and thoroughly developed. In either case, the best and cheapest line for a given ruling grade having been determined, additional surveys will develop the fact that an additional expenditure of so much will permit a certain reduction in the ruling grade. To put it in figures, the ruling grade of the existing road or of the proposed route is 1.4 per cent; the existing traffic is 10 trains per day each way which will be affected by the rate of the ruling grade; it is found possible by an expenditure of $50,000 to reduce the rate of the ruling grade to 1.2 per cent. Will it pay? The method of answering this question will be developed in the following sections.

192. Determination of the Rate of the Ruling Grade. Many railroads have been constructed without any preliminary considerations of limiting grade. The road was laid out where convenient, the cuts and fills limited by some vague ideas as to the maximum which is thought desirable, but without apparent regard for the resulting grade. The logical result would be some one maximum grade at one place and a gradually increasing number of lesser grades. Since in such a case, all one-engine train loads are limited by the one maximum grade, there is usually an unquestionable policy in cutting down this one grade to the limit of the next lower grade. If there are several cases of the next lower grade, all must be lowered if at all. Each successive lowering involves a larger number, and usually a limit to practicable reduction of grade is soon reached unless a very radical change in the whole route is considered. The determination of the proper rate for the ruling grade requires the best judgment of an experienced engineer. A general outline of the method is as follows:

The determining points of the route are first selected and their relative elevation and their distance apart is obtained. The ratio of these figures for any stretch gives a first approximation of the through grade. The most unfavorable stretch is then studied to see whether the grade may be reduced there by development. When some one place is very much worse than any other place, the best solution may be at once obtained by operating this one grade as a pusher grade and then reducing all other grades to the corresponding through grade. It may thus be seen that there are infinite possibilites in the selection of a ruling grade, in the routes which must be followed in order to obtain such grades, and in the amount of money which may profitably be spent. When the choice has narrowed down to two routes with two limits of grade, the better of which will cost a definite sum more than the other, then the advisability of spending the extra sum may be investigated as will be developed.

193. Maximum Train Load on any Grade. The load which any locomotive may haul up a grade is absolutely limited by the weight on the drivers. The adhesion of the drivers to the rails depends somewhat on the condition of the rails, whether wet or dry, and is also modified by the use of sand. One-third is about the maximum adhesion obtainable and even this can hardly be depended on. Under very unfavorable conditions the adhesion will drop to one-fifth and even less, but by the use of sand an adhesion of one-fourth can always be obtained and this can generally be relied on as the adhesion unaided by sand. Therefore this value will be used. The *net* load behind an engine is the difference between the *gross* load which can be hauled by a given weight on the drivers and the weight of the engine itself. The almost infinite variety of weights and proportions of locomo tives makes it impracticable to tabulate the hauling capacity of every variety of locomotive, but in Table XIX are given the figures for several locomotives which may be considered as typical of American practice for both narrow and standard gauge.

For example, the "consolidation" engine given in the last line of the table has a weight on the drivers of 181,200 pounds. At one-fourth adhesion the tractive force is 45,300. If we assume 6 pounds per ton as the tractive resistance on a level, the total resistance on a 1.0 per cent grade will be 6+20 pounds or 26 pounds. Dividing 45,300 by 26 we have 1,742, the total possible weight of the train in tons. This is the gross load. The net load is 1,742 – 162.4 = 1,579.6 tons. If this train was made up entirely of 100,000-pound capacity cars, weighing when loaded 134,000 pounds or 67 tons, the train could consist of 23 cars and have 38 tons left over for the caboose and for margin. By a very simple application of the above figures, the great advantage of a reduction of grade is readily seen. If the grade were cut down to 0.85 per cent, a reduction of less than 8 feet in a mile, the total tractive resistance would be reduced to 23 pounds per ton. This would increase the net tonnage by 228 tons, which is more than the weight of three of the 50-ton cars, or an increase of over 14 per cent.

TABLE XIX.

Tractive Power of Various Types of Locomotives at Various Rates of Adhesion.

Kind.	Gauge.	Total weight of engine and tender.		Weight of engine only	Weight on drivers.	Tractive power when rate of adhesion is		
		lb.	tons.			$\frac{1}{4}$	$\frac{9}{40}$	$\frac{1}{5}$
American.....	Nar.	49,000	24.5	32.000	22.000	5.500	4.950	4.400
"	"	80.000	40	49.000	32.000	8.000	7.200	6.400
Mogul	"	81.000	40.5	51.000	42.000	10.500	9,450	8,400
10-wheel.......	"	87,000	43.5	55.000	42.000	10.500	9.450	8.400
Consolidation	"	62.000	31	39.000	34.000	8.500	7.650	6.800
"	"	144,000	72	94,000	84.000	21.000	18,900	16.800
American ...	Stand.	104.000	52	62,000	40.000	10.000	9.000	8,000
"Chautauqua"	"	314,600	157.3	190,600	99.400	21.850	22,365	19.880
Mogul	"	206.000	103	126.000	106.400	26.600	23,940	21.280
10-wheel	"	276,000	138	176.510	127.010	31.752	28.577	25.402
Consolidation	"	214.000	107	120.000	106.000	26.500	23,850	21.200
"	"	324,800	162.4	204,800	181.200	45,300	40,770	33.240

Table XX is very useful in these computations. It shows in the two parallel columns the relations between grades in "rate per cent" and "feet per mile." Then, on the mathematical basis that the grade resistance per ton of 2000 pounds on any grade equals 20 times the rate of grade in per cent, the total resistance

TABLE XX.

Total Train Resistance Per Ton (of 2000 Pounds) on Various Grades.

Grade.		When tractive resistance on a *level* in pounds per ton is					Grade.		When tractive resistance *on a level* in pounds per ton is				
Rate per cent.	Feet per mile.	6	7	8	9	10	Rate per cent.	Feet per mile.	6	7	8	9	10
0.00	0.00	6	7	8	9	10	2.00	105.60	46	47	48	49	50
.05	2.64	7	8	9	10	11	.05	108.24	47	48	49	50	51
.10	5.28	8	9	10	11	12	.10	110.88	48	49	50	51	52
.15	7.92	9	10	11	12	13	.15	113.52	49	50	51	52	53
.20	10.56	10	11	12	13	14	.20	116.16	50	51	52	53	54
0.25	13.20	11	12	13	14	15	2.25	118.80	51	52	53	54	55
.30	15.84	12	13	14	15	16	.30	121.44	52	53	54	55	56
.35	18.48	13	14	15	16	17	.35	124.08	53	54	55	56	57
.40	21.12	14	15	16	17	18	.40	126.72	54	55	56	57	58
.45	23.76	15	16	17	18	19	.45	129.36	55	56	57	58	59
0.50	26.40	16	17	18	19	20	2.50	132.00	56	57	58	59	60
.55	29.04	17	18	19	20	21	.55	134.64	57	58	59	60	61
.60	31.68	18	19	20	21	22	.60	137.28	58	59	60	61	62
.65	34.32	19	20	21	22	23	.65	139.92	59	60	61	62	63
.70	36.96	20	21	22	23	24	.70	142.56	60	61	62	63	64
0.75	39.60	21	22	23	24	25	2.75	145.20	61	62	63	64	65
.80	42.24	22	23	24	25	26	.80	147.84	62	63	64	65	63
.85	44.88	23	24	25	26	27	.85	150.48	63	64	65	66	67
.90	47.52	24	25	26	27	28	.90	153.12	64	65	66	67	68
0.95	50.16	25	26	27	28	29	.95	155.76	65	66	67	68	69
1.00	52.80	26	27	28	29	30	3.00	158.40	66	67	68	69	70
.05	55.44	27	28	29	30	31	.05	161.04	67	68	69	70	71
.10	58.08	28	29	30	31	32	.10	163.68	68	69	70	71	72
.15	60.72	29	30	31	32	33	.15	166.32	69	70	71	72	73
.20	63.36	30	31	32	33	34	.20	168.96	70	71	72	73	74
1.25	66.00	31	32	33	34	35	3.25	171.60	71	72	73	74	75
.30	68.64	32	33	34	35	36	.30	174.24	72	73	74	75	76
.35	71.28	33	34	35	36	37	.35	176.88	73	74	75	76	77
.40	73.92	34	35	36	37	38	.40	179.52	74	75	76	77	78
.45	76.56	35	36	37	38	39	.45	182 16	75	76	77	78	79
1.50	79.20	36	37	38	39	40	3.50	184.80	76	77	78	79	80
.55	81.84	37	38	39	40	41	4.00	211.20	86	87	88	89	90
.60	84.48	38	39	40	41	42	4.50	237.60	96	97	98	99	100
.65	87.12	39	40	41	42	43	5.00	264.00	106	107	108	107	110
.70	89.76	40	41	42	43	44	5.50	290.40	116	117	118	119	120
1.75	92.40	41	42	43	44	45	6.00	316.80	126	127	128	129	130
.80	95.04	42	43	44	45	46	6.50	343.20	136	137	138	139	140
.85	97.68	43	44	45	46	47	7.00	369.60	146	147	148	149	150
.90	100.32	44	45	46	47	48	8.00	422.40	166	167	168	169	170
1.95	102.96	45	46	47	48	49	9.00	475.20	186	187	188	189	190
2.00	105.60	46	47	48	49	50	10 00	528.00	206	207	208	209	210

in pounds per ton is found by adding to the grade resistance 6, 7, 8, 9 or 10 pounds per ton according to the basis used. By dividing the total train resistance in pounds per ton into the total adhesive force of the drivers we have the gross weight of the train.

Examples. **1. What will be the total tractive resistance on a 1.35 per cent grade, that on a level being 8 pounds per ton?** Ans. 35 pounds.

2. How much of a load can a standard-gauge mogul engine (as given in Table XIX) draw up a 0.95 per cent grade, assuming a rate of adhesion of $\frac{9}{40}$ and a tractive resistance on a level of 8 pounds per ton? Ans. Tractive power, 23,940; tractive resistance per ton, 27 pounds; gross load = 23,940 ÷ 27 = 887 tons; net load = 887–103 = 784 tons.

To solve problems for engines not given in Table XIX, use the general rule: The maximum load behind an engine on any grade may be found by multiplying the weight on the drivers by the ratio of adhesion and dividing this by the sum of the grade and tractive resistances per ton; this gives the gross load, from which the weight of the engine and the tender must be subtracted to find the net load.

194. Proportion of Traffic Affected by the Ruling Grade. The ruling grade is that which limits the length of trains. There are many roads which do not have a traffic so large that any of their trains are absolutely limited by their weight. This is especially true of passenger trains, since, as before stated, it is wise to increase the number of trains on light traffic roads as soon as the traffic will permit, and before the trains get crowded, so as to encourage traffic by the increase in facilities. It is perhaps true that on most roads the passenger trains are altogether unaffected by the rate of the ruling grade; *i. e.*, it would be possible, so far as the engine is concerned, to add one more passenger car to the train without stalling the train on any grade. The same statement is generally true with respect to the local freight trains, especially when the traffic is so small that mixed trains are run.

In computing the saving by reducing the ruling grade, it must also be considered that even though there are some passenger trains which run as many cars as one engine may haul, a reduction in grade would not be utilized by reducing the number of passenger trains to handle a given number of cars. The reduction in the traffic facilities would react too seriously on the passenger traffic. Therefore the only advantage which could be claimed for the passenger traffic from a reduction of the rate of the ruling grade would be just those advantages which result from the reduction of any grade, the reduction in fuel consumed, or the possible increase in speed, etc., advantages which are apart from the limiting effect of the grades. The limiting effect also applies only to traffic in one direction, or at least the grades and traffic in the other direction are wholly separate matters.

As discussed later, the ruling grades in the two directions may or may not be equal. The only favorable effect that the reduction of any ruling grade can have on traffic moving in the opposite direction is a slight saving in the use of brakes, the amount of which is so insignificant that it may be ignored, especially as it has absolutely no relation to the matter of limiting the train length. Therefore it is generally true that except on heavy traffic roads the only trains to be affected by the rate of the ruling grade are those which handle freight in car-load lots. This applies to all trains which are special in character such as coal trains, ore trains, and all those who handle such a bulk of any one product that the cars are loaded to their full capacity and the cars are grouped into trains according to the engine capacity. Such conditions are the most favorable for economic operation as it insures the minimum of cost for the handling of a given volume of traffic.

In the following calculations on the saving by reducing the ruling grade the character of the traffic must be closely studied, and although short light passenger trains should not be included, care should be taken that a fair estimate should be made regarding the medium weight trains.

195. Cost of an Additional Train. The gross receipts for handling a given volume of traffic is a definite gross amount whether it requires one train or two, four trains or five. The cost of running a train one mile is approximately a constant quantity whether the train is heavy or light. If the cost were exactly constant, the saving in operating expenses would be strictly in proportion to the number of trains saved. Assuming a given amount of freight to be handled, or rather that a given number of cars are to be handled, how much will be saved by handling them in nine trains rather than ten, or four rather than five, or sixteen rather than twenty? To put it the other way, what will be the extra cost of one additional train? Note that the total number of cars to be handled is the same. The additional cost is that of running an extra engine under the given circumstances.

Maintenance of way. Since the number of cars using the tracks is the same, any additional wear and tear on the track is that due to the extra engine. It is estimated that about 25 per cent of the expenses of repairs of roadway (item 1) vary with the tonnage. It is also estimated that a locomotive, on account of the greater concentrated weight on its drivers, is responsible for 50 per cent of track repairs, in spite of the much greater gross tonnage of the cars. On this basis, 50 per cent of 25 per cent, or 12.5 per cent, is allowed for item 1, and 50 per cent for all other items except 5 and 7, which may be considered as unaffected.

Maintenance of equipment. Whether the higher or lower ruling grade be chosen, the trains are supposed to be loaded to the maximum capacity of the engine on the ruling grades. But when the heavier ruling grade is chosen, the trains are correspondingly lighter and therefore the stress on the engine on all lighter grades, which may constitute 90 per cent of the whole, will be much less. Although the engine repairs on the extra engine used would be as much as any of the others, the saving on all by the reduction of their work on the easier grades would accumulate to quite a figure. The saving on each engine would evidently be greater when four is increased to five than when nineteen is increased to twenty. But the accumulated saving for one engine would be approximately a constant, which has been estimated as 20 per cent, or the net addition to the cost of engine repairs for the additional engine would be only 80 per cent of the average charge.

On a similar basis there is a saving in car repairs, but since the number of cars used is constant, it produces an actual saving which must be tabulated *negatively*. The average drawbar pull will be less, and this form of repairs will evidently be less, and the stresses due to stopping and starting will be less with a corresponding reduction in such repairs. This has been computed to amount to –5 per cent for passenger cars (item 13) and –10 per cent for freight cars (item 14). Items 15 and 19 will evidently be but little affected if at all, and therefore no allowance will be made for them.

Conducting Transportation. Items 20, 21, 26 to 32, 35 to 37, 45 and 46 are to be considered as varying directly as the mileage and 100 per cent is allowed on all those items. While there is some reason for considering that some portion of some of the items varies in accordance with the ton mileage rather than with the train mileage, the effect of this will be very slight and will compensate for a slight increase in other small items on which nothing has been allowed. Engine supplies (items 22 to 25) will increase nearly as the mileage, but there will be a saving on the consumption on each engine for the same reason as stated for item 12. The engines will use less fuel and water while running over the easier grades of the road. This saving has been computed as 25 per cent of the consumption by a single locomotive, or the net

TABLE XXI.
Cost of an Additional Train to Handle a Given Traffic.

No.	Item (abbreviated)	Normal average.	Per cent. affected.	Cost in per cent.
1	Roadway	10.779	12.5	1.25
2–4	Rails, Ties, Bridges, etc.	6.992	50	3.50
5	Fences, etc.	.557	0	0
6	Buildings, etc.	2.096	50	1.05
7	Docks and wharves	.256	0	0
8–10	Miscellaneous	.525	50	.26
	Maintenance of way	21.205	—	6.16
11	Superintendence	.631	0	0
12	Repairs—locomotives	6.258	80	5.01
13	" —passenger cars	2.218	–5	–.11
14	" —freight cars	7.048	–10	–.70
15–19	Miscellaneous	1.488	0	0
	Maintenance of equipment	17.643	—	4.20
20	Superintendence	1.759	100	1.76
21	Enginemen	9.640	100	9.64
22–25	Fuel, etc	11.162	75	8.37
26–32	Train and station service, etc	23.481	100	23.48
33–34	Car mileage. Hire of equipment	2.200	0	0
35–37	Damages, etc	1.848	100	1.85
38–44	Miscellaneous	5.439	0	0
45–46	Stationery, and other expenses	1.195	100	1.20
	Conducting transportation	56.724	—	46.30
47–56	General Expenses	4.428	0	0
		100.000		56.66

RAILROAD TUNNEL AT LOOP IN TEHACHIPI PASS, SOUTHERN CALIFORNIA

On line of Southern Pacific Railroad.

addition to the total consumed by the use of the extra engine would be 75 per cent of the average charge. Car mileage (item 33) and hire of equipment (item 34) will be unaffected; also items 38 to 44. The general expenses are of course unaffected.

As before, we may consider this percentage of cost as cents per mile. But there is the additional charge for the capital cost of the extra engine. The cost of maintenance repairs and renewals is included in the above estimate, but the total first cost divided by its estimated mileage life should be added for each mile run. Locomotives cost anywhere from $5,000 to $15,000, but considering $10,000 as an average figure and the mileage life as 800,000 miles, this makes an extra charge of 1.25 cents per mile, or a total of 57.91 cents, which we may call 58 cents.

196. Numerical Example. A road is being operated with ruling grades of 1.80 per cent (95.04 feet per mile). There are six daily freight trains and two daily passenger trains which are affected by the rate of the ruling grade. Surveys have proved the feasibility of reducing the ruling grade to 1.45 per cent by an expenditure of $300,000. Will it pay? The engines used on freight work are "consolidation" engines of the type mentioned in Table XIX, weighing 214,000 pounds, with 106,000 pounds on the drivers; when the ratio of adhesion is $\frac{9}{40}$, the tractive force is 23,850; with a tractive resistance on a level of 8 pounds per ton, the total resistance on the 1.8 per cent grade is 44 pounds per ton and the gross load per train is 23,850 ÷ 44, or 542 tons, and the net load is 435 tons. The six freight trains can therefore handle 2,610 tons.

Although it was said that two passenger trains are affected by the ruling grade, it would be manifestly impracticable and unwise to cut down the number. To them the reduction of the grade can only be utilized to increase, if necessary, the number of cars in a train to handle a hoped-for increase in traffic. But such a problem is outside of the present computations and therefore we must ignore the passenger trains except to consider that such trains have been helped and their expenses reduced, but that the reduction of expense is outside of a limiting effect such as is considered in this problem. On the 1.45 per cent grade the total tractive resistance, when that on a level is 8 pounds, is 37 pounds per ton. The gross load for those same engines = 23,850 ÷ 37 = 645 tons; the net load is tons which is something over one-fifth of the 2,610 tons of the o r calculation. This shows that the reduction in grade will permit the hauling of the same (and even a little greater) traffic in five trains instead of six. If the division is 100 miles long, the annual saving would be

$$100 \times 365 \times 0.58 = \$21,170.$$

Capitalized at 5 per cent, this equals $423,400. Since the work could be done for $300,000, it would evidently be justifiable to do it, if the necessary capital can be raised, for the computed annual saving does not include the advantages of the grade reduction to the two heavy passeng referred to above, nor to the considerable number of lighter hich will run on that grade. Nor does it consider the fact th raffic of almost any road will grow and that the advant re ly proportional to the traffic. If during the next fifteen ars the traffic doubles (as is not unlikely), the advantages a rtionately greater.

PUSH ES.

197. General Principles r Economy. It frequently happens that the natural line of a road includes a few grades which are considerably higher than all rades. These higher grades may be practically hopeless, because a material reduction in them would cost more than it is worth, or more than the general financial condition of the road can afford. A common error is to consider such a grade as the ruling grade and then recklessly permit, at any other place on the road, the adoption of any grade less than this on the ground that it could never limit the operation of trains. But in such cases, it *may* be easily practicable to operate the higher grades with a pusher engine and cut down all lesser grades to such a rate of grade that the "through" engine can haul as many cars on them as two engines can haul on a pusher grade. The economy underlying the method may be seen by a simple illustration which is freed from all details.

Assume that on a division 100 miles long there are two grades of five miles each on which pusher engines are to be used; assume that the grades on the other 90 miles are so low that one engine may haul as many cars on them as two engines can haul on the pusher grades; then by using pusher engines the weight of all heavy trains may be doubled and the heavy freight may be handled in half as many trains. But this economy is effected at the cost of operating the pusher engines. Using single engines for the whole trip, it will require 200 engine miles to haul a double load. But a single engine can haul the double load over 90 miles of the run, and the same engine with one pusher can haul it up the heavy grades. Each pusher engine will travel 10 miles on each grade, or twenty miles for the two, and the total number of engine miles for a double load will be 120, instead of 200. And when it is considered that the cost of a pusher engine mile is far less than that of an ordinary train mile (as will be shown), the advantages of the method are still more marked.

Of course the full economy of the method is only realized when the maximum *through* grade bears its proper relation to the pusher grade. If the maximum through grade is greater than its proper corresponding value for the pusher grade, then the number of cars is limited by that through grade and the power of the pusher engine is not completely utilized on the pusher grade. Economy of operation requires that an engine should work nearly to the limit of its capacity for as large a portion of the time as possible, and therefore when a heavy engine is compelled to haul a light train over nine-tenths of the route in order that there shall be sufficient power on the other tenth, where alone it is needed, it indicates a lack of economy in the design. It now becomes necessary to develop the proper relation between through and pusher grades.

198. Balance of Grades for Pusher Service. This will be easiest understood by a numerical problem. Suppose that at two or three places on the line it seems impracticable to obtain at a reasonable expense a less grade than 2.10 per cent (nearly 111 feet per mile). But since it seems practicable to make a very much lower grade elsewhere, we will compute the corresponding through grade as the grade to work for. Assume that the through and pusher engines are alike and that they are the kind given in Table XIX, consolidation type, weighing 214,000, with 106,000 on the drivers. With $\frac{9}{40}$ adhesion the tractive power is 23,850, and when the tractive resistance on a level is 8 pounds, the total resistance on the 2.10 per cent grade is 50 pounds per ton.

The gross load for a single engine is 477 tons and the net load is 370 tons. Two such engines could handle 740 tons on that grade. If that net load were to be hauled by a single engine, the gross load would be 847 tons. Dividing the tractive power (23,850) by the gross load (847) we have 28.1 as the possible tractive force per ton. But under the same conditions as above this is the force required on a 1.0 per cent grade, which is therefore the corresponding through grade, or the rate to which all minor grades should be reduced. This may likewise be considered the ruling grade of the road, the 2.10 per cent grade being equally a ruling grade using a pusher. Sometimes, although rarely, two pusher engines are used. This requires the solution of the proportional pusher grade for one pusher and the proportional through grade. A justification for this policy in the above case would be the fact that although many of the grades could not with reasonable expenditure be reduced much under 1.5 per cent, a very large proportion of the road could be kept at less than 0.7 per cent. The precise determination is as follows:

Tractive power of three engines $= 3 \times 23850 = 71{,}550$ lbs.

Resistance on 2.10 per cent grade $= 8 + (20 \times 2.1) = 50$ pounds per ton.

$71550 \div 50 = 1431$ = gross load in tons.

$1431 - (3 \times 107) = 1110$ = net load in tons.

$1110 + (2 \times 107) = 1324$ = gross load in tons on the one-pusher grade.

Tractive power of two engines $= 2 \times 23850 = 47{,}700$ lbs.

$47700 \div 1324 = 36.03$ = possible tractive force in pounds per ton.

$(36.03 - 8) \div 20 = 1.40\%$ = permissible grade for one pusher.

$1110 + 107 = 1217$ = gross load on the through grade.

Tractive power on one engine $= 23{,}850$ pounds.

$23850 \div 1217 = 19.60$ = possible tractive force in pounds per ton.

$(19.60 - 8) \div 20 = 0.58\%$ = permissible through grade.

It should at once be realized that under the assumed conditions, which are perfectly normal, the above combination of through, one-pusher, and two-pusher grades are precisely what would be required and that each one is equally a limiting grade under the conditions. But those relative grades are mathematically precise only for that particular type of engine and for that particular ratio of track adhesion and that particular normal track resistance. Fortunately the variations are not great. A storm would increase the resistance and would probably decrease the adhesion by making the rails slippery. Although this would materially reduce the train load, it may be demonstrated that these variations would have opposite effects in altering the ratios, and therefore that the ratio of through to pusher grade would remain substantially the same.

TABLE XXII.

Balanced Grades for One, Two, and Three Engines.

Basis.—Through and pusher engines alike; consolidation type; total weight, 107 tons; weight on drivers, 53 tons; adhesion, $\frac{9}{40}$, giving a tractive force for each engine of 23,850 lb.; normal track resistance, 6, (also 8) lb. per ton.

Through grade.	Track resistance, 6 lb.			Track resistance, 8 lb.		
	Net load for one engine in tons (2000 lb.).	Corresponding pusher grade for *same net load.*		*Net* load for one engine in tons (2000 lb.).	Corresponding pusher grade for *same net load.*	
		One pusher.	Two pushers.		One pusher.	Two pushers.
Level.		Per cent.	Per cent.		Per cent.	Per cent.
Per cent.	3868 tons	0.28	0.55	2874 tons	0.37	0.72
0.10	2874 "	0.47	0.82	2278 "	0.56	0.98
0.20	2278 "	0 66	1.08	1880 "	0.74	1.23
0.30	1880 "	0.84	1.33	1596 "	0.92	1.47
0.40	1596 "	1.02	1.57	1384 "	1.09	1.70
0.50	1384 "	1.19	1.80	1218 "	1.27	1.92
0.60	1218 "	1.37	2.02	1085 "	1.44	2.14
0.70	1085 "	1.54	2.24	977 "	1.60	2.36
0.80	977 "	1.70	2.46	887 "	1.77	2.56
0.90	887 "	1.87	2.66	810 "	1.93	2.76
1.00	810 "	2.03	2.86	745 "	2.09	2.96
1.10	745 "	2.19	3 06	688 "	2.24	3.15
1.20	688 "	2.34	3.25	638 "	2.40	3.33
1 30	638 "	2 50	3.43	594 "	2.55	3.51
1.40	594 "	2.65	3.61	555 "	2.70	3.68
1.50	555 "	2.80	3.78	521 "	2.85	3.85
1.60	521 "	2.95	3.95	489 "	2.99	4.02
1.70	489 "	3.09	4.12	461 "	3.13	4.17
1.80	461 "	3.23	4.27	435 "	3.27	4.33
1.90	435 "	3.37	4.43	411 "	3.42	4.49
2.00	411 "	3.52	4.59	390 "	3.55	4.63
2.10	390 "	3.65	4.73	370 "	3.68	4.78
2.20	370 "	3.78	4.88	352 "	3.81	4.92
2.30	352 "	3.91	5.02	335 "	3.94	5.05
2.40	335 "	4.04	5.15	319 "	4.07	5.19
2.50	319 "	4.17	5.29	304 "	4.20	5.32

A variation in the type of engine and in the ratio of total weight to weight on the drivers will also alter the ratio of through and pusher grades, but a table such as Table XXII computed for a consolidation engine will serve very well for almost any consolidation engine. Such a table will do for preliminary computations, especially when the road is not yet constructed and the precise weight of the engines is still unknown. For the reconstruction of an old line, a table similar to Table XXII should be compiled, based on the most exact and reliable data obtainable for the particular case. The table is derived in precisely the same manner as the case worked out above.

Example. Assume a through grade of 60 feet per mile (= 1.14 per cent), what is the corresponding one pusher grade? Interpolating in Table XXII, we find that when the normal track resistance is 6 pounds per ton, we find a net load of about 722 tons and a pusher grade of 2.25 per cent. If the track resistance was increased to 8 pounds per ton, the maximum load would be reduced to 668 tons, and the corresponding pusher grade becomes 2.30 per cent. The difference is only 0.05 per cent or less than 3 feet per mile. It may seem anomalous that an increase in resistance would permit a higher pusher grade, but this is due to the fact that the computed net load is much less—so much less that a larger proportion of the tractive energy is available for overcoming grade.

100. Operation of Pusher Engines. Economy in pusher engine work demands that the schedule of trains be so arranged that the pusher engine can be kept constantly at work. If there are several short pusher grades separated by several miles of level track, it either means that a pusher engine must be assigned to each grade, where there may not be enough work to keep it busy, and therefore its daily cost divided by its engine mileage is abnormally large, or else it must travel uselessly over some intervening stretches of level track in order to be at hand when wanted. Even the time table of the trains must be arranged with reference to the pusher service so that there will not be an accumulation of traffic at the pusher grades at certain hours, with nothing to do at other times. The locating engineer has no concern with the operation of trains, but he should bunch the pusher grades if possible, even spending a little additional money for it if necessary.

On very light traffic roads, where the trains are so few that the method does not interfere with the schedule, a pusher grade may be operated by a single engine, by taking half of the train up first, leaving it on a switch, and then returning after the other half. This means slow time (which a very poor road can afford); it means a saving of the cost of the extra engine, and also the comparatively costly maintenance of it if the total amount of pusher work is very light. Such a road is probably not blessed with an excess of traffic except during a small part of the year, and the cost and maintenance of a useless pusher for a large part of the year is thereby saved. But it should be noted that even if it is expected to follow this policy, it does not make the slightest difference in the design of the pusher grades or in the ratio of through to pusher grades.

Another possible method of economizing on pusher service, especially on light traffic roads, where a pusher grade begins or ends near a station yard which is so large that a switching engine is necessary for at least part of the day, is to combine the switching and pusher work. A little ingenuity in planning the schedule will thus enable the pusher engine to utilize its whole time in useful work.

200. Length of a Pusher Grade. - The true length which must be considered in the following calculations is always somewhat in excess of the length of the actual grade as measured on the profile. Although it is sometimes possible, by having the pusher engine approach the train from behind, to accomplish its work without stopping the train either at the top or bottom of the grade, yet this requires an extra length of track and a considerably extra mileage on the part of the pusher engine. For passenger service the assistant engine is always placed in front, and although it is practicable to uncouple the assistant engine at the top of the grade, run it ahead at increased speed, run it on a siding and again clear the main track without stopping the train, it is usually necessary to stop the train at the bottom to couple on. Increased mileage is necessary for this. The stoppage and restarting of a heavy train uses up as much energy as would carry the train several miles and therefore an increased run by the pusher engine is justifiable if it will save stopping the train. A siding at or near the bottom and top of the grade (and also a telegraph office) is a convenience and almost a necessity for the quick and safe operation of pusher grades, and while they must be clear of the grade it is sometimes more convenient to remove them some distance from the ends of the grade. Each case is a separate problem, but the length to be used in the following calculations must always be the actual run of the pusher engine, which will be somewhat in excess of the actual length of the pusher grade as shown on the profile.

201. Cost of Pusher Engine Service. The cost depends partly on the work done by the engine and partly on mere time. The wages of the enginemen must be paid on a per diem basis rather than on a mileage basis, and if the engine does not run many revenue miles, the cost for the miles it does run is increased. The items of maintenance of way will evidently be increased in the same proportion as was computed in §195 for the additional cost of another engine. The only item in maintenance of equipment to be affected is that of engine repairs, for which we must allow the full 100 per cent. The wages of enginemen, the cost of all engine supplies, the wages of switchmen, etc., and the expenses of telegraphing will all be increased in their full proportion, but it is difficult to see how any of the other items can be increased except in special cases. Collecting the items for which a charge is made, we have Table XXIII.

TABLE XXIII.

Items of the Cost Per Mile of Operating a Pusher Engine.

No.	Item (abbreviated).	Normal average.	Per cent affected.	Cost per engine mile, per cent.
1	Repairs of roadway..........	10.779	12.5	1.35
2-4	Rails, Ties, Bridges, etc......	6.992	50	3.50
6	Buildings, etc................	2.096	50	1.05
8-10	Miscellaneous	.525	50	.26
12	Repairs of locomotives......	6.258	100	6.26
21	Enginemen..................	9.640	100	9.64
22-25	Engine supplies..............	11.162	100	11.16
28	Switchmen, etc	4.057	100	4.06
29	Telegraph expenses..........	1.906	100	1.91
	Total			39.19

As before, we will consider this percentage as cents per mile, but we must add 1.25 cents per mile to allow for the capital cost. This makes a total of 40.44 cents per engine mile. But since each pusher engine must run two miles to do one mile of pusher engine service, we may say that each mile of pusher grade will cost 80.88 cents (say 81) for each train assisted.

202. Numerical Problem. In §196 is given the problem of a road with ruling grades of 1.80 per cent which could be reduced to 1.45 per cent grades at an expenditure of $300,000. This would permit the hauling of six train loads with five engines. An alternative solution is here suggested for discussion. Assume that the 1.8 per cent grades occur along two stretches having "pusher grade lengths" (see §200) of 7 miles and 8 miles. From Table XXII we see that the through grade corresponding to a one-pusher grade of 1.8 per cent, track resistance 6 pounds, is 0.86 per cent. Assume that it is found that all the other grades (outside of the pusher grade stretches) can be reduced to this limit of 0.86 per cent by an expenditure of $200,000. How will this plan compare with the other? This will at once permit cutting down those six trains to three. The saving on one train is $21,170, and on three is $63,510. But this is only accomplished by using pusher engines on fifteen miles of pusher grade each day on each of three trains. This will cost 3 × 15 × 365 × 0.81 = $13,304.25. Apparently this makes a saving of about $50,000 per year to offset the immediate expenditure of $200,000.

There is one defect in the above solution. Its magnitude can only be determined by a closer knowledge of the attendant circumstances. The above pusher engine service calls for only 3 × 15 × 2 = 90 running miles by the pusher engines. The pusher grades are perhaps many miles apart, and therefore one pusher engine must be stationed at each grade. This gives but 45 miles of running per day for each. This is so small that a much larger charge than 81 cents should be made. But even if the charge were made twice as much, which is probably more than ample, the saving would still be $37,000 per year, which would pay the interest on three times the assumed cost.

As the number of trains requiring pusher engine assistance increases, the calculation on the above basis would grow more accurate and it may be considered as correct when each pusher engine accomplishes a mileage of from 150 to 200 miles. On the above conditions, the utilization of the 1.80 per cent grade as a pusher grade and the reduction of the lower grades seems to be the better method, for the cost of accomplishing it would be less and the annual saving would be greater; but other conditions could easily reverse this decision.

BALANCE OF GRADES FOR UNEQUAL TRAFFIC.

203. Fundamental Principles. The volumes or weights of the traffic in each of the two directions on any road are usually quite different and frequently that in one direction is four or five times that in the other. The number of through engines passing over the road in each direction each day is necessarily equal, and, unless some engines run "light", the number of trains must be the same. The number of passenger cars must be the same, and, in the long run, even the number of freight cars must be the same. But if the weight of the freight is very largely greater in one direction than in the other, the cars will run nearly or quite full in one direction and nearly or quite empty in the other direction. The lightly loaded trains will therefore weigh less and, with the same through engine, can surmount a steeper grade than the heavily loaded train. It therefore becomes justifiable to introduce a slightly heavier grade against the lighter traffic, if economy of construction is thereby obtained.

There are many roads which are not concerned with this phase of grade. When a branch line runs to some terminus in the mountains so that practically all of the heavy grades are in one direction and there are no opposing grades when running out of the mountains (barring a few harmless sags), there is no limitation of trains by grades except in the one direction, and there is no necessity or object in computing any balance. But the through trunk lines, especially those running east and west, find that their east-bound traffic is three or four times their west-bound traffic.

As a single instance, from 1875 to 1880, the ratio of the east-bound ton mileage to the west-bound on the Pennsylvania railroad was more than 4.5 to 1. The difference of elevation of the termini has little or no importance in this case since it is so small compared with the total length of the line, and since any possible effect which it might have had on the grade is utterly lost in the heavy grades in both directions when crossing the mountains. Admitting the justification of a variation in the ruling grade in opposite directions so as to produce a virtual equality in tractive effort, it now becomes necessary to compute the theoretical balance.

204. Computation of Theoretical Balance. In spite of the very evident disparity in the weight of the freight traffic in the two directions, there are some equalizing factors, as will be shown.

1. The locomotive and passenger car traffic in the two directions are equal.

2. The passenger traffic in the two directions will be equal. There is a slight exception to this when a road handles a considerable number of emigrants, but the effect of this is absolutely insignificant, especially in view of the further fact that the ratio of dead load to live load is very high with passenger traffic. Considering that even 50 passengers in a car, assumed to weigh 150 pounds apiece, would only weigh 7,500 pounds which is but $\frac{1}{6}$ of the 45,000 pounds which the car probably weighs, even a considerable variation in passenger traffic each way would not affect the gross load materially.

3. Empty cars have a greater resistance per ton than loaded cars. The difference will amount to about two pounds per ton. Therefore, although a train of loaded cars will require a greater gross tractive effort than an equal number of empty cars, the ratio will not be in proportion to the gross tonnage.

4. In spite of the best care and regulations on the part of the traffic department, many freight cars will run in the direction of the heaviest traffic either empty or but partly loaded.

5. In general it is the freight which has the greatest bulk and weight, such as grain, coal, lumber, ore, etc., which is run from the rural districts toward the cities and manufacturing districts.

6. The return traffic, which consists chiefly of manufactured products, and which is worth as much as the other, weighs but a small fraction of the other.

As a simple numerical illustration, assume that it has been determined that on a given east and west line the east-bound traffic is three times the west-bound traffic. Some of the recent 100,000 capacity cars weigh only 25 to 30 per cent of their total loaded weight, but considering the other cars of which the conditions are not so favorable and the partly loaded cars which run even in the direction of heavy traffic, we will assume that 40 per cent of the gross load is dead weight and that 60 per cent is live load for the east-bound traffic. Using the same cars, the live load for west-bound traffic is $\frac{1}{3}$ of 60 per cent or 20 per cent of the gross east-bound tonnage, and the gross train load is 40 per cent plus 20 per cent or 60 per [illegible] the gross east-bound tonnage. Assume that the ruling grad[illegible] [illegible]nst the east-bound traffic has been fixed at 1.2 per cent. [illegible] [illegible]ld be the corresponding ruling grade against the west-[illegible] [illegible]raffic? From Table XXII we find that with a track [illegible]e on a level of 6 pounds per ton the *net* load behind a con-[illegible]ation engine of the type there mentioned would be 688 tons on a 1.2 per cent grade. Of the 688 tons, 40 per cent, or 275 tons, [illegible] be the weight of the cars, and 413 tons the weight of the load. [illegible] the west-bound trip the live load will be one-third of [illegible] [illegible]s [illegible] 138 tons. The cars weigh 275 tons, but two-thirds of [illegible]em, or [illegible], are empty, a[illegible] tractive resistance on a level [illegible]wo pounds [illegible]er ton more or [illegible] pounds per ton. The tractive resistance on a level is therefore

183	tons of cars	@ 8	pounds per ton	=	1,464	pounds
92	" " "	@ 6	" " "	=	552	"
138	" live-load	@ 6	" " "	=	·828	"
107	" engine	@ 6	" " "	=	642	"
520	"				3,486	"

Subtracting 3,486 from 23,850, the total adhesion of that engine, we have 20,364, the total force available for grade. Dividing this by the total load 520, we have 39.16 pounds per ton which will overcome a grade of 1.96 per cent. Under the above conditions this is the correct figure. Of course the value would be slightly modified by using a different type of engine, but a numerical trial will show that the variation is very small for any reasonable variation in the type of engine, or the other minor conditions. The great cause of variation is the ratio of traffic in the two directions.

205. Estimation of the Relative Traffic. The estimation of the relative volumes of traffic on a road yet to be constructed is usually a matter of sheer guesswork except as it might be inferred from existing roads which are similar in character. But this problem often forms one of the features of the plans for the improvement of existing lines and in such a case there is an abundance of existing data. Since it concerns only the ruling grades, it affects only those trains which are affected by the rate of the ruling grade. It is unfortunately true that the fluctuations of traffic are such that a ratio which might have been perfect at the time of its computation may become considerably in error for a long period of time if not permanently. A change in the development of the country may turn an agricultural region into a manufacturing region or the discovery of vast deposits of coal or ore may result in a considerable and permanent change in the flow of traffic.

The Interstate Commerce Commission report for 1901–2 gives the following as the chief items of freight tonnage:

Bituminous coal	255,792,615 tons
Lumber	88,653,853
Ores	84,866,105
Anthracite coal	83,720,175
Grain	52,257,455
Stone, sand and other like articles	52,281,827
Coke	47,451,880
Merchandise	47,015,216
30 other headings	
Total	1,061,618,627 tons.

It will readily be seen that the above items only include the heavy bulky freight. Such an item of manufacture as agricultural implements only weighed 2,445,955 tons, and household goods and furniture but little more. These figures emphasize the statements made above regarding the relative weights of various classes of traffic.

Perhaps the safest general rule is to say that opposite ruling grades should be made equal unless there is a definite reason for making them unequal. The reconstruction of the great trunk lines, on which so much money is now being spent, invariably aims at a lower grade against east-bound traffic than that allowed against the west-bound. The Canadian Pacific railroad had scarcely been completed before there was a reconstruction with this end in view. While it is one of the most uncertain elements to calculate, its justification under certain conditions is unquestionable.

THE IMPROVEMENT OF OLD LINES.

206. Classification. The improvements here considered are chiefly those which deal with changes of alignment. The replacement of antiquated station buildings with modern structures, the reconstruction of bridges to meet the demands of heavier rolling stock, the substitution of heavier rails, and even the installation of block signaling are improvements which are of great value if not matters of absolute necessity. Some of them demand a high order of engineering talent for their proper installation, but they are not matters which directly affect the alignment.

(*a*) *Increasing the business.* It has been previously stated that increasing the business done by the road is the most important element to be considered, since a comparatively small change in the business done suffices to change profit to loss or vice versa. To some extent this matter is under the control of the engineer, insofar as he is able to select the location of the road as it approaches towns. Many roads have ruined their business possibilities in a town by the false economy of running the line through the outskirts rather than boldly forcing a way to the heart of the business portion. Many a road has ultimately perceived its blunder and has retrieved it by spending millions where tens of thousands would have accomplished the same result during the original construction.

Usually the engineer is not to blame for these blunders. He is not permitted to decide questions of such magnitude. But there are lesser questions such as the crossing of a considerable river to reach a town, when the best route for the road (topographically) is on the farther bank. The cost of such an improvement is a mere matter of constructive estimate. The advantages to be gained are very uncertain and perhaps variable. Wellington estimates that the location of the station one mile from the center of population will cut off 25 per cent of the traffic, even when it is non-competitive. A handicap of one mile in competing for traffic would make such competition almost hopeless. There are numerous instances in this country where two lines of road compete for the business of a string of towns. The road with the convenient stations not only does the most of the business—it does nearly all of it.

(*b*) *Changes in curvature, and distance and the elimination of sags and humps.* The financial value of these changes has already been discussed. Usually any change of alignment will involve some of each of these three classes of changes. Although the changes should be favorable in all respects, if possible, yet it may readily happen that a great advantage in grade (for example) might only be obtained by a relatively insignificant increase in curvature. In such a case, the variation of all these elements must be computed separately and their *net* effect considered.

(*c*) *Relocation of stations and passing sidings.* The relocation of stations is sometimes advisable in order to give greater facilities and therefore attract new business. It is also done when an enlargement is necessary which has become impracticable at the old location. Another very important reason is found when the station is located on a heavy grade. The same principles apply to the location of passing sidings. The tractive resistance to starting is so great that if a station or passing siding is located on a grade which is equal to (or even much less than) the ruling grade, the total resistance when starting a heavy train may be so great that it may limit the weight of trains to less than that computed for the ruling grade. Under such circumstances a relocation or reconstruction of some sort is almost imperative. This important kind of improvement will be further discussed.

(*d*) *Increasing the train load.* Some of the methods of doing this have already been discussed under "Grade". The special application of those methods to constructed lines will be treated later.

207. Advantages and Disadvantages of a Relocation. All calculations on the reconstruction of an old line involve the cost of the improvement and the financial or operating advantages to be gained by it. The first element is merely an estimate of constructive cost, which is always obtainable as closely as is necessary. The advantages to be gained are especially uncertain when the business done is unknown. But in the case of an operated road, the business done is a perfectly definite quantity. Even the cost per train mile, which may differ very largely from the average values which have been used in the previous calculations, is readily determinable, and of course these values should be used. Calculations involving grade also involve the tractive power of the locomotives used. In previous calculations, average values were necessarily used. On a constructed and operated road, the actual capabilities of the locomotives in operation may be easily studied and very accurate calculations based on them.

On the other hand, a reconstruction of even very small stretches of the line involves the abandonment of work on which much money has been spent and also involves the expenditure of further sums to obtain an advantage which may seem very clear and cer-

tain to the engineer, but which may seem very uncertain to the non-technical board of directors. It should not be forgotten that the cost of a road to subgrade is frequently not more than one-third of the total cost of construction and equipment, and that in some cases an expenditure of a small percentage of the total cost will suffice to make a change which will permit an increase in the train load of 50 per cent or more. And yet the fact that the money spent in the original construction of the abandoned portion must be considered as sunk (unless the old right of way may be utilized for some other purpose) will have a deterrent effect on the directors.

It should not be forgotten that there is a legal question involved in the abandonment of an old line. It may be detrimental to the business interests of a factory which was originally located alongside of the railroad for the special purpose of transportation facilities. The injury to that factory's interests would be considerable and might be the ground for a successful suit for damages. This element of cost of the whole project must therefore be considered.

REDUCTION OF THE VIRTUAL GRADE.

208. Collection of Data for Computations. A comparatively simple series of observations showing the actual handling of trains on the ruling grades of a road has frequently demonstrated that a mere change in operating methods, without any constructive change in the roadbed, will suffice to make possible an increase in the train load. The first step in the investigation is to locate all grades which are steep enough or long enough so that there is any possibility that they may be ruling grades. The original profiles of the road will suffice for this when they are available. Near the bottom (and also near the top) of all such grades a pair of targets should be erected at some measured distance apart so that the velocity of the train may be determined by noting the time required to pass that distance. On the one hand, that distance should be made very short so that there will be no appreciable change in velocity while running that distance, but, on the other hand, when it is very short it is difficult to obtain the time with a close *percentage* of accuracy. The distance recommended by Wellington is 293 feet 4 inches, which is exactly $\frac{1}{18}$ of a mile. Then

MOUNTAIN RAILWAY OPENING ITS WAY INTO THE WILDERNESS

A scene in the State of Washington.

if s = the time in seconds required to pass this space of 293 feet 4 inches, the velocity in miles per hour = 200 ÷ s.

As a numerical illustration, assume that the time required to pass that space is exactly 10 seconds. If $\frac{1}{18}$ of a mile is run in 10 seconds, the mile would require 180 seconds, which is three minutes, and the speed is 20 miles per hour. But 200 ÷ 10 = 20, the required velocity. If a stop watch is used which will record speed to fifths of a second, a ten-second run is recorded to 2 per cent. The faster the speed, the less the percentage of accuracy. If the speed of the train is liable to be much greater, which is very probable in these days of high-speed freight trains, a better distance will be 440 feet. Then the velocity in miles per hour = 300 ÷ s. This will give a better percentage of accuracy. The velocity in miles per hour for any other length may be obtained from the formula

$$V = \frac{(\text{length of base in feet}) \times 3600}{(\text{time in seconds}) \qquad \times 5280} \quad \textbf{(106)}$$

The targets may be anything which is readily visible when riding past; even a piece of white cotton cloth, tacked on to the fence, will answer the purpose. Since one of the essential questions is to determine whether the engine is doing all that it can or should, the conditions under which the engine is working must be known. Although economy and efficiency of working require that the steam be used expansively when running at high speeds, yet the maximum tractive force is obtained from the locomotive when it is running at rather slow speed, with its maximum steam pressure, wide-open throttle and cutting off nearly if not quite at full stroke. Since a slippery track will influence the tractive force, a note of the condition of the track should be made, as well as the force and direction of the wind or any other circumstance which would have such influence that it could not be considered a normal test. The weight of the train should also be obtained to the nearest ton. This may be done by noting the dead weight of the cars and engine and also obtaining from the freight department the loading of the cars.

After having located the targets, several trips should be taken over the line on all trains which are so heavy that they are affected by the ruling grades. It will be necessary to note the velocity, not only when climbing up a ruling grade, but also when descending into a sag and when the acquired momentum is essential to ascending the succeeding up grade. Several trips should be made on trains which are working under nominally the same conditions as to type of engine and train load. A comparison of results will show the general accuracy of the work, while an average of the results will give values which should be very reliable.

209. Utilization of the Data. A little consideration of the laws of mechanics as applied to the motion of trains will show the accuracy of the following statements:

(a) If, on a uniform grade, the velocity increases, it shows that under those conditions of engine working, the load is less than the engine *can* handle on that grade.

(b) If the velocity decreases when the engine is working at its maximum capacity, it shows that the load is greater than the engine can handle on an indefinite length of such a grade. It shows that the grade is being operated by momentum. From the rate of the decrease of the velocity, the maximum practicable length of such a grade, starting with a given velocity, may be easily computed.

(c) By combining the results for any one train and engine when the steam pressure and other working conditions are as nearly uniform as possible, but when the train is climbing different grades, it will be possible to compute values of the normal tractive resistance, since the grade resistance and variations in kinetic energy are definite mathematical quantities.

(d) Having an independent value for the normal tractive resistance on that weight of rail and condition of roadbed, and the power of the locomotives in actual use on that road, reliable estimates may be made of the effect of changes of grade.

The force required (in addition to all the tractive and grade resistances) to impart to a train a given velocity in a given distance may be computed as follows: What force P is required to impart to a weight W a velocity v in a distance s? By an amplification of the laws of Mechanics, as given in §183, we may say that the kinetic energy of a body, which $= W\frac{v^2}{2g}$, also equals Ps.

From which, we may write $$P = \frac{Wv^2}{2gs} \qquad (107)$$

As a corollary to this, we may also write that, if P′ = the force required to increase the velocity of W from v_1 to v_2, then

$$P' = \frac{W}{2gs}\,(v_2^2 - v_1^2) \qquad \mathbf{(108)}$$

Substituting in this formula W = 2,000 pounds (one ton), g = 32.16, and s_0 = 5,280 feet (one mile) and also multiplying by $\left(\frac{5280}{3600}\right)^2$ so as to change the velocity unit from feet per second to miles per hour, we have

$$P' = .01267\ (V_2^2 - V_2^2).$$

Increasing this coefficient 5 per cent to allow for the rotative kinetic energy of the wheels, as explained in §183, we have

$$P' = .0133\ (V_2^2 - V_1^2) \qquad \mathbf{(109)}$$

In equation 109, *P′* is the force required per ton (in addition to all the other resistances) to increase the velocity of the train from V_1 miles per hour to V_2 miles per hour and to accomplish that increase in a distance of *one mile*. Since it may be more convenient to state the distance in which the increase in velocity must be accomplished in *feet*, we may write

$$P'' = \frac{70.224}{s}\,(V_2^2 - V_1^2) \qquad \mathbf{(110)}$$

Example. Assume that two bases of 440 feet each have been laid off at the top and bottom of a 1.70 per cent grade, the distance between the bases being 6,000 feet. A train with cars weighing 435 tons drawn by a consolidation engine weighing 107 tons is timed as it runs up the grade. The time on the lower base is $16\frac{1}{5}$ seconds and on the upper base $13\frac{3}{5}$ seconds.

$$\frac{300}{16.2} = 18.5 \text{ and } \frac{300}{13.6} = 22.1,$$

which are the respective velocities in miles per hour. It is at once apparent that the train has gained velocity on the grade, and since this proves that it is doing more work than is required by the grade and tractive resistances, the engine is not loaded to its full capacity.

Substituting 18.5 = V_1 and 22.1 = V_2 in equation 110, with s = 6,000, we deduce P″ = 1.70 pounds, which is the additional pull which is actually being developed by the engine for each ton's weight of the train. The total surplus pull = 542 × 1.70 = 921 pounds. A precise calculation of the additional loading which could be hauled by this surplus power requires the determination, by a comparison of a series of such observations as the above, of the normal tractive resistance, but for our present purpose it is sufficiently accurate to use the average value of 8 pounds per ton. On this basis, since the grade resistance is 34 pounds per ton on a 1.70 per cent grade, the total resistance is 42 pounds per ton and the increased loading could be 921 ÷ 42 = 22 tons. This is a small margin, less than the weight of a loaded car, although it is possible in this case that one more car might be hauled up this grade by reducing the train velocity and thereby reducing the train resistance.

Of course the above pair of observations and their numerical computation must be considered as but one of a very large number which should be taken on that particular grade. All trains which may possibly be affected by the rate of the ruling grade should be timed and the capacity of each engine studied. Of course there will be considerable variation in results which should nominally be the same, the variation being due to changes in conditions, slippery rails, imperfections in the roadbed which increase the resistance, etc. The method is capable of giving results from which very positive deductions may be drawn.

210. Reducing the Starting Grade at Stations. The required drawbar pull when starting a train is very much greater per ton than when running at a moderate velocity. This is due partly to the fact that the journal resistance is so increased that the mere tractive pull, regardless of grade and inertia, is perhaps 10 pounds per ton greater. And then the additional force required to impart a given velocity in a given distance is very considerable. If a train is required to attain a velocity of 25 miles per hour in a distance of 1,500 feet, starting from rest, the additional force per ton may be computed from equation 110 by substituting 1,500 for s, 25 for V_2, and 0 for V_1. Solving, we obtain $P'' = 29.26$ pounds. If we add 16 pounds per ton for the resistance at the instant of starting, we have 45.26 pounds, which is the equivalent of a 2.26 per cent grade or over 119 feet per mile. And this would be the virtual grade even if the actual grade were level. If, in addition to this, a considerable grade must be climbed, the power of the locomotive would be insufficient unless the load was very small. Of course the practical result would be that the train would take a much greater distance in which to acquire its velocity, and in the case of a freight train, it probably would not acquire that speed of 25 miles per hour on any considerable up grade.

In §§183–186 is shown the economic value and even necessity of cutting down the rate of grade, especially if it be a ruling grade, when a stop for heavy trains occurs on that grade. Of course this should be done during the original construction of the road. But when this precaution has been neglected or when there arises the unforeseen necessity for a stopping place on a steep grade, some change of grade is almost imperative. This may sometimes be accomplished by a change which extends beyond the limits of the original grade and which permits an actual reduction of grade at the station.

Another possible plan is the insertion of a hump, such as is shown in Fig. 161. Assume that on an original uniform grade of 1.2 per cent a stop becomes necessary at B. A train passing A at a speed of 20 miles per hour would have a "velocity head" of 14.05 feet. If a hump of 10 feet were made at the point B by introducing a 1.45 per cent grade for a distance of 4,000 feet, the train would still have a velocity head of 14.05 — 10 = 4.05 feet, which corresponds to a speed of 10.7 miles per hour, when it reached B.

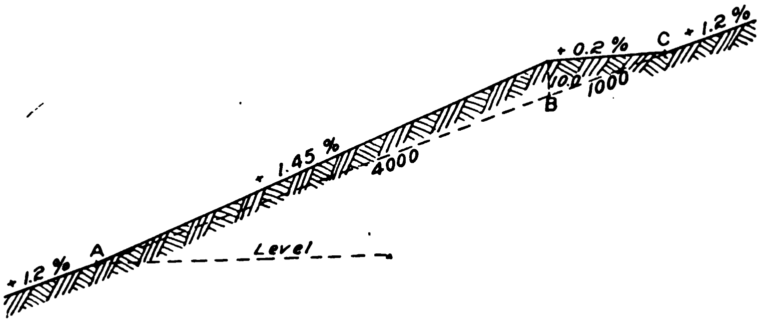


Fig. 161.

Even at a velocity of 17 miles per hour at A there would be sufficient kinetic energy in the train to climb the 10 feet. The work done by the engine as it reaches A is 24 pounds per ton for grade and perhaps 8 more for tractive resistances. Assume that the run-off from B to C is made 1,000 feet long. Even this will leave the grade + 0.2 per cent. This will require 4 pounds per ton for grade and say 16 more for excessive starting resistances. But as the velocity increases this starting resistance will be speedily reduced to perhaps 6 pounds per ton. Taking the average figure of 11 and adding 4 for grade we have 15 pounds per ton resistance. Subtracting this from 32, the total power of the engine, we have left 17 pounds per ton available for creating kinetic energy. Transforming equation 110, by considering V_2 as the unknown quantity, and calling $V_1 = 0$, we have

$$V_2^2 = \frac{P''s}{70.224} \qquad (111)$$

Substituting 17 for P'' and 1,000 for s, we find that $V_2^2 = 242$ and $V_2 = 15.6$, the attainable velocity in miles per hour when the train arrives at C.

Probably the velocity would be somewhat in excess of this since the tractive resistance would very quickly reduce down to 6 pounds per ton or even less, the power then available for kinetic energy would then be greater and the desired velocity could then be acquired more quickly. It is quite possible or probable that the above solution could be profitably modified to better suit the detailed conditions, but it is given to illustrate the general principle of a feasible method of solving such problems.

The effect of the hump on trains running down grade and stopping at B is favorable rather than otherwise. The virtual up grade from C to B (although it is actually a down grade) will assist in stopping the train at B. The increase in the down grade from B to A will enable the train to attain its normal velocity with a decrease of time and energy.

INCREASING THE WEIGHT OF THE RAILS.

211. Essential Qualities of a Rail. A rail reduces the tractive force required to haul a wheel having a given load. If a wheel is being hauled over soft sand, it sinks to a considerable depth into the sand, and the effect is virtually the same as if it were being hauled up a considerable grade. The work done includes that required to compress the sand and make the rut which is thus formed. If the wheel were rolled on a surface-like rubber, although the tractive force includes the work necessary to compress the rubbery surface to some depth, yet there would be some compensation owing to the forward component of the pressure against the wheel tread as it occurs back of the center. Such a component is wholly lacking on a sand road.

Such resistances are reduced to exceedingly small quantities on a rail. The rails rest on ties and the ties on ballast and the resistance to pressure depends largely on the tamping of the ballast. Any permanent deformation of the ballast means just so much energy wasted. This is largely reduced by the strength and stiffness of the rail. The very high elasticity of the rail surface reduces the energy spent in compressing that surface by the wheel to an insignificant quantity. It may therefore be said that when we buy rails, what we want are the imponderable qualities of *strength* and *stiffness* rather than mere tons of steel. If the rails do not have the requisite strength and stiffness, they are correspondingly worthless, no matter how much they weigh. Although the sections of rails of varying weights are not geometrically similar figures, they are so nearly so that the following demonstration is substantially true:

The laws of mechanics tell us that if prismatic beams have geometrically similar cross-sections, the *strength* varies as the *cube* and the *stiffness* as the *fourth power* of the homologous dimensions. But since the area of the sections (and therefore the weight of the rails per foot or yard) varies as the *square*, it follows that the strength varies as the $\frac{3}{2}$ power and the stiffness as the square of the weight. For all ordinary variations in weight the price per ton is constant.

212. Economics of Heavier Rails. As a numerical illustration of the above, assume that the road is laid with 70-lb. rails. Replacing them with 80-lb. rails will add 14 per cent to the cost, but it will also add 22 per cent to the strength and nearly 31 per cent to the stiffness. The increase in weight per mile of single track will be 15.71 gross tons, which at $30 per ton would cost $471.30 per mile. One mile of the 70-lb. rails would cost $3,300. At the same rate per unit of stiffness the 80-lb. rails are worth $4,309,80, an increase of $1,009.80, which is obtained at an expenditure of $471.30. At the same rate per unit of strength the 80-lb. rails are worth

$4,032.60, an increase of $732.60, which is obtained at an expenditure of $471.30. Of course this line of reasoning might be carried to the extent of attempting to justify the use of, say, 100-lb. rails on a very light traffic road. An expenditure is not justifiable simply because it is cheaply obtained. The real question becomes—is the increase in weight and resulting cost justified by a sufficient decrease in operating expenses?

A precise analysis would be very difficult if not impracticable. Stronger and stiffer rails decrease tractive resistance and therefore decrease coal consumption, wear and tear of rolling stock and even the wear and tear of the track, owing to a decrease in destructive impact and vibration. A decrease of only one pound per ton would increase the possible train load on a level by 10 to 15 per cent. The saving of one pound per ton would have less and less importance as the grade grew higher. Since tractive resistance is largely dependent on the tamping of the ballast and a multitude of other influences, of which the strength and stiffness of the rail is but one, it is impracticable to compute the true value of these rail characteristics.

An approximate rule, given by the Baldwin Locomotive Works says, "Each ten pounds weight per yard of ordinary steel rail, properly supported by cross ties (not less than 14 for a 30-foot rail), is capable of sustaining a safe load per wheel of 2,240 pounds." A 100,000 pound capacity car will weigh when loaded about 140,000 pounds or 17,500 pounds per wheel. By the above rule, roads handling such cars should have rails weighing about 80 pounds per yard. The heaviest consolidation engine referred to in Table XIX has 181,200 pounds on the drivers or 22,650 pounds per driver. By the above rule such an engine should have 100-lb. rails to run on. And even this does not mean that even heavier rails might not be advisable and economical even with the lighter loads. It is probably true that it is economical to use a rail which is heavier than that size which is merely "safe."

REVIEW QUESTIONS.

PRACTICAL TEST QUESTIONS.

In the foregoing sections of this Cyclopedia numerous illustrative examples are worked out in detail in order to show the application of the various methods and principles. Accompanying these are examples for practice which will aid the reader in fixing the principles in mind.

In the following pages are given a large number of test questions and problems which afford a valuable means of testing the reader's knowledge of the subjects treated. They will be found excellent practice for those preparing for College, Civil Service, or Engineer's License. In some cases numerical answers are given as a further aid in this work.

REVIEW QUESTIONS

ON THE SUBJECT OF

PLOTTING AND TOPOGRAPHY

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1. What scale of map and what general method of field work would you adopt for surveying and plotting the smaller details of topography? Give reasons.

2. Define a contour. What is a contour interval and what conditions determine it?

3. Describe "hachures" and the principles on which they are drawn.

4. How would you determine contours on a small area if only a tape, rod, and hand-level were available? Explain why.

5. The elevations of two adjacent points are 68.2 and 87.9 feet respectively; the points are 90 feet apart horizontally; locate the five-foot contours between them.

6. Vertical angles taken to the 6-foot and 1-foot marks on a rod held vertically gave the angles $-3^\circ 23'$ and $-4^\circ 07'$ respectively;
    (a) what is the true horizontal distance from instrument to rod?
    (b) what is the relative elevation of the ground at the rod and the telescope?
    (c) how much would the horizontal distance be altered by a change of one-half minute in either vertical angle?

7. What are the disadvantages of the gradienter which do not apply to the stadia method?

8. What are the limitations of the stadia method?

9. What are the three shortened methods of "reducing" stadia observations?

10. What are the true horizontal distances and differences of elevation for the following observations when $(f' + c) = 1.0$

t? Compute according to the strictest theory and also accord- to the approximate method,

Rod interval	1.69	2.65	7.80
Vertical angle	18° 15′	6° 10′	1° 10′

11. Describe the process and make the computations for tting the angles 12° 30′, 27° 20′ and 62° 40′ by the tangent thod.

12. Rule a specimen of profile paper for a profile 2,500 feet and 100 feet range of elevation, at the scales 400 feet = 1 horizontally and 20 feet = 1 inch vertically. Plot a profile ording to the following notes:

a.	Elev.	Sta.	Elev.	Sta.	Elev.	Sta.	Elev.	Sta.	Elev.
150	189.	155	215.8	160	230.6	165	200.5	170	187.6
151	193.5	156	219.0	161	226.8	166	194.2	171	193.8
152	198.9	157	224.6	162	221.2	167	180.6	172	198.3
153	204.6	158	230.2	163	214.8	168	172.3	173	204.5
154	210.5	159	231.3	134	206.3	169	179.4	174	210.0

lot a grade line which has an elevation of 196.5 at Sta. 150 and a grade of + 0.3%.

13. Discuss the proper choice of a protractor for plotting angles.

14. Draw the parallels and meridians as given in § 21, page 34, by the method of trapezoidal projection, described on page 28. Draw it on tracing paper or tracing cloth and lay over the solution called for in § 21, so as to note the discrepancies of this method.

15. Describe the method of making a "simple conic projection"; what portion of the resulting map is correctly represented? What is the nature of the distortion of the other portions?

16. What determines the radius of curvature of any developed "parallel" on the "polyconic" system?

17. What are the applications and limitations of the "three-point" problem?

18. Describe the three general methods of plotting the position of a point determined by the three-point method.

19. What three colors are chiefly used for topographical maps; classify the topographical features which are indicated by each color?

REVIEW QUESTIONS

ON THE SUBJECT OF

RAILROAD ENGINEERING

PART I

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1. What are the elements of railroad location which are in general antagonistic? Deduce from the above the chief duties of the locating engineer.

2. What is the chief object to be accomplished by a reconnoissance survey?

3. What are the three elements involved in the survey of any line and what are the methods of determining these elements in reconnoissance surveys?

4. What is the practical value and what are the limitations of barometric leveling?

5. What is the general object to be accomplished by means of a preliminary survey?.

6. To what extent should the compass needle be used during preliminary surveys?

7. Why is it that a Locke level with its limited accuracy is a proper instrument for cross-section work?

8. Under what circumstances is the stadia method advantageous for preliminary surveys?

9. What is the justification of making two or more preliminary surveys through difficult portions of the route?

10. How are the tangents and curves for the "location" determined? How is the location survey "tied" to the preliminary survey?

11. How would you select a low-grade line through a difficult piece of mountainous country?

12 How are transit stations and bench marks secured against disturbance during construction of the road?

13. Define the degree of a curve. What is the approximate rule for the radius of a curve of a given degree? What is the percentage of error of this rule for a 10° curve?

14. What is a sub-chord? What will be the excess length of a sub-chord with a nominal length of 45′ on a 5° curve?

15. What is the difference between the nominal length of a railroad curve and its true length measured on the arc? What will this amount to in the case of a 6° curve subtending a central angle of 34° 30′?

16. If two adjacent tangents which make an angle of 24° 16, are connected by a 3° 30′ curve, what will be the distance from the vertex to the point of the curve?

17. In the case given above, how far will the curve pass from the vertex?

18. A 3° 30′ curve is to begin at Sta. 142 + 65 and is to have a total central angle of 28° 30′; compute the deflections from the tangent at the P.C. to each station and to the P.T.

19. In the above case assume that on account of obstructions to sighting it was necessary to set up the instrument at Sta. 147 and sight back to Sta. 144. Applying the rule of section 25, what should be the reading of the horizontal plate when the instrument is sighted at Sta. 144, and what should be the reading when it is sighted ahead at Sta. 148?

20. Assume that a 3° curve having a central angle of 14° 30 is to be located by tangential offsets; make a sketch of this case and compute and mark on the sketch the deflections and distances.

21. After running a 4° curve to some point *n* as in Figure 16, the curve is found to be obstructed. It is estimated that the curve would again be clear about 400 feet further on. Compute the long chord *nm* and the angle which *nm* would make with a tangent to the curve at *n*. What would be the offset from this long chord to the second station beyond *n*?

22. Give detailed solutions of the problems stated in section 30?

23. Give detailed solutions of the problems stated in section 34?

24. What is the essential character of a transition curve and why it is necessary?

25. Two tangents make an angle of 34° 25′. They are to be joined by a 5° curve having a 1°-per-25-foot spiral with 4 chords at each end of the curve. Compute the distance from the vertex of the curve to the point of beginning of the spiral and also the distance from the vertex to the middle point of the curve.

26. Write out the detailed solution of the problem given in section 43.

27. A grade of – .9 per cent is changed in a sag to a grade of + 1.3 per cent. Assume that these grades will be joined by a vertical curve 1,100 feet long. The vertex of the curve occurs at Sta. 45 + 20 and has an elevation of 122.7. Compute the elevations for each station point in the length of this vertical curve.

28. What are the usual slope ratios for excavation in ordinary earth work; for a fill of ordinary earth; also for a fill of blasted rock?

29. What width of roadbed is usually required for cuts and fills for both single and double track.

30. Discuss the importance of a good drainage of the roadbed.

31 Why is it that the computed volume of any mass of earthwork may not represent accurately the amount of earth actually excavated even though the mathematical computations have been made with accuracy? What points of a cross-section are determined in earthwork surveys and why?

32. Describe in your own words, illustrating the description by an assumed numerical case, the method of locating slope stakes.

33. What is meant by level sections? Why are they probably inaccurate?

34. Under what circumstances are computations made on the base of level sections especially inaccurate?

35. What are the advantages and the errors of the method of equivalent sections?

36. Why is it easier to compute the volume of a prismoid by "averaging in areas" and applying a "prismoidal correction" rather than by applying the prismoidal formula exactly?

37. Verify and write out the detailed determination of all quantities in the numerical example of section 58, which are not already worked out in that section.

38. Sketch a cross-section having two intermediate points beside the slope stake on each side of the center. Indicate the distance of each point out from the center and its height above the level of the roadbed by a letter, then write out an expression for the area which shall be similar to equation 46 by a strict application of the rule printed in italics in section 60.

39. State the approximate rule for determining the prismoidal correction for a section of earthwork having "irregular" end sections.

40. Verify in detail the determination of the quantities in the tabular form for reducing field notes in section 62 which are opposite "Sta. 45."

41. What are "borrow pits?" What is their usual form and what is the method of computing the volume?

42. Why is it that there is usually a "correction for curvature" in earthwork computations? Under what conditions is the correction zero?

43. What is the approximate method of computing the eccentricity of the center of gravity of a section when the section is "irregular?"

44. Describe how you determine the eccentricity of the center of gravity of a "side hill section."

45. What is the fundamental difference between a detonating and a slow-burning blasting powder?

46. How much blasting powder should be used in a blast-hole when the line of least resistance is 8 ft.? How much dynamite would serve the same purpose?

47. What are the general methods of hand drilling? What method of hand drilling should be adopted in a tunnel heading where rapidity of work is of great importance?

48. Why is tamping important?

49. What is the usual method of exploding blasting charges?

50. Discuss the shrinkage of earthwork embankments. What methods are adopted to make earth embankments permanent?

51. What is the usual classification of excavated material and how are the different classes distinguished?

52. What is the basis of the financial importance of accurate tunnel surveying?

53. Describe the distinguishing features of underground surveys.

54. What is the method of transferring distance, elevation and true direction from the surveys to the bottom of a shaft?

55. What principles underlie the design of the cross-section of a tunnel?

56. What grade should be employed in a tunnel and what are the practical limitations?

57. What should be the cross-section of a timber-lined tunnel and why?

58. Wherein lies the practical difficulty in constructing a portal of a tunnel?

59. What are the advantages and disadvantages of pile trestles?

60. What are the advantages and disadvantages of the different methods of pile driving?

61. How much is the safe load that can be placed on a pile which is driven by a steam hammer, the hammer weighing 5,500 lb. and having a fall of 40 inches when the penetration due to the last blow is 1½ inches?

62. What is a mortise joint (in trestle building) and what are its advantages?

63. Describe some of the features of multiple story construction of trestles.

64. Describe the methods of making temporary foundations for trestles.

65. Why are trestle stringers usually made of comparatively high but narrow timbers?

66. What are "separators" in trestle work and why are they used?

67. What are corbels and why are they used?

68. What are the functions of guard rails on trestles and how should they be located with reference to the traction rails?

69. Describe, using a sketch, what you consider the best method of elevating the outer rail on a trestle and give your reasons for the use of that method?

70. What features govern the choice of wood in trestle construction?

REVIEW QUESTIONS

ON THE SUBJECT OF

RAILROAD ENGINEERING

PART II

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1. What is the chief cause of the deterioration of locomotive boilers due to impure water supply?

2. What are the chief difficulties encountered in the construction of engine houses and how are the difficulties met?

3. What elements must be considered in computing the total cost of any kind of railroad tie?

4. What is the preferable method of locating ties with reference to rail joints?

5. Assuming that an 85-pound rail and a 70-pound rail have similar cross-sections, what is the relative stiffness?

6. What are the elements of a perfect rail joint and why is it impossible to produce a perfect rail joint for steam railroad work?

7. Why are plain smooth spikes preferable to spikes which are jagged?

8. What are the three principles which form the basis of the design of nut locks?

9. Give a brief statement of the general methods of obtaining a pure water supply.

10. What are the elements of an ideal form of ballast? What the disadvantages of "mud" ballast? What are the advantages of stone ballast?

11. What are the causes, other than mere decay of the wood, which require that ties should be renewed?

12. What are the features of the A. S. C. E. rail section which are constant for all weights of rail and what are the proportions which are constant or nearly constant?

13. What are the advantages and disadvantages in using very long rails?

14. What are the advantages obtained by the use of tie plates?

15. How many track bolts in a mile of single track using six-bolt splice bars and 30-foot rails?

16. How much is allowed for rail expansion and how is this practically provided for?

17. How much gap would you allow at a rail joint when the temperature of the rail at the time of laying is 45° F?

18. What should be the middle ordinate of a 30-foot rail bent to a 40° curve?

19. What should be the superelevation of the outer rail for a 60° curve when the maximum speed is 45 miles per hour?

20. If the maximum speed for trains is assumed at 60 miles per hour, what will be the length of a string or tape which, when stretched as a chord inside the rail, will give a middle ordinate equal to the required superelevation?

21. What is the fundamental advantage of a point switch over a stub switch?

22. Suppose it were required to make to order a frog having a frog angle of 6° 30′; what would be the frog number?

23. Verify the calculations for the length of the lead of a switch from a straight track using a No. 8 frog on the basis, first, of circular lead rails, and second, of straight point rails and straight frog rails, using the values given in Table III.

24. If a No. 8 frog has been used in switching from a straight track, what will be the radius of the connecting curve when the distance between track centers is 13 ft.?

25. What will be the length and radius of the connecting curve running from a switch on the outside of a main track, which is a 4° 30′ curve, the frog used being No. 9 and the distance between the track centers 13 ft.?

26. Make all the computations for the location of a turnout to the inside of a 4° curve using a No. 8 frog.

27. What are the different kinds of tracks making up a freight yard?

28. By what device is engine service economized in planning a freight yard?

29. What is the basic principle of the absolute block system of signaling? What is permissive blocking?

30. What are distant signals? How do they facilitate train speed?

31. What is the principle of automatic signal systems?

32. What are the arguments for and against semaphores as compared with enclosed signals?

33. How much will be the change of length of a signal wire or pipe which is 1,200 feet from the signal tower for a change of temperature from 120° F. to 0°? How will such a change of length be neutralized in the compensating mechanism?

34. By what device will an electro-pneumatic signal be set at danger if there should be a failure of either the electric current or the pneumatic pressure?

35. Upon whom must the responsibility rest in the case of a collision in a yard operated with interlocking switches and why?

36. In what way will the careless use of the track gauge give inaccurate results and why?

37. What is the danger involved in the use of track jacks?

38. Under what conditions is it economical to provide sleeping and dining accommodations for the work-train gang?

39. Under what conditions is the ditching work of a work-train gang abnormally expensive?

40. What is the most economical method of distributing ties for renewals?

41. Under what conditions is it cheaper to load ballast from a gravel pit by hand labor?

42. Under what conditions is it cheaper to fill up trestle sites rather than rebuild the trestles?

43. What precautions must be taken in refilling high trestles?

44. What are the qualifications for a successful roadmaster?

REVIEW QUESTIONS

ON THE SUBJECT OF

RAILROAD ENGINEERING

PART III

1. Discuss the two classes of financial interests which are concerned in the ownership of railroads—indicating the security and profits of each.

2. Describe methods of estimating the probable volume of traffic on a proposed road, indicating particularly the factors which must be taken into consideration in making such a calculation.

3. Discuss the division of the gross revenue and the percentages which are ordinarily spent in operating expenses, fixed charges, and dividends.

4. Discuss operating expenses per train mile; their uniformity for heavy and light traffic roads; the tendency toward variation of the chief items.

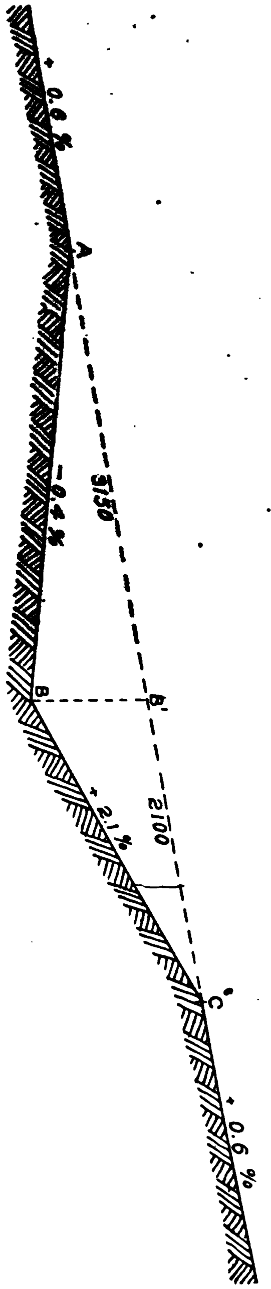
5. Discuss the items of expense per train mile which are affected by a small increase in distance.

6. By the cutting of a tunnel through a sharp bow in the line of a railroad following a river bank, the total length of the line would be shortened 2.6 miles. What would be the justifiable expenditure that might be made for the construction of the tunnel, money being obtainable at 5 per cent, and the traffic of the road being 16 trains per day each way.

7. Discuss the items of expense per train mile which are affected by curvature.

8. Assume that the elimination of the bow-line in the railroad along the river (see Question 6) will cut out 174° of curvature. How much further expenditure for purposes of construction would this item justify?

9. Discuss curve compensation; the reasons for its use; the values which should be employed.



10. Explain the distinction between minor and ruling grades.

11. What is the meaning of "velocity head"? What is the velocity head of a train when moving at the following velocities in miles per hour: 21; 27.4; 32.25? What velocities correspond to velocity heads of 18.58; 38.92; 49.25?

12. Explain the fundamental principle of a virtual profile, and describe its use and possible misuse.

13. Classify "minor grades" on the basis of their effect on operating expenses.

14. Discuss the effect on operating expenses of a sag or a hump on a grade line.

15. Compute in detail, giving reasons for the various steps, the operating value of filling up the sag A B C in the accompanying illustration, assuming that there are nine heavy freight trains and four heavy passenger trains per day each way.

16. Discuss the distinctive elements of a ruling grade.

17. An "Atlantic" type passenger engine, with loaded tender weighs 273,000 pounds, the weight on the drivers being 84,000 pounds. How many tons of cars can it haul up a 1.70 per cent grade when the tractive resistance on a level is 7 pounds and when the tractive adhesion is $\frac{1}{4}$?

18. Discuss the elements of the cost of an extra engine to handle a given gross number of loaded cars.

19. How much may justifiably be spent to reduce the rate of a ruling grade from 1.7 per cent to 1.0 per cent when

there are 10 trains per day operating against those grades? Assume that the trains are hauled by consolidation engines weighing 324,000 pounds, with 181,200 pounds on the drivers; assume 6 pounds tractive resistance on a level. Give all the details of the computations and state the reasons for each step.

20. Demonstrate the fundamental principle in the economy of pusher grades.

21. Given a maximum grade of 2.10 per cent, what would be the corresponding through grade if *one* pusher engine is used — the engine being of the type described in Question 19? If *two* pushers were used on the 2.10 per cent grade, what would be the corresponding one-pusher and through grades?

22. Discuss the elements of the cost of the operation of pusher engines.

23. With the data of Problem 19, assume that the 1.7 per cent grade is to be treated as a pusher grade. Compute the corresponding through grade and then compute the value of such a method on the basis that there are three pusher grades of 4, 6, and 7 miles.

24. Discuss the fundamental principle of the "balance of grades for unequal traffic".

25. Describe the method of making speed observations for a study of the capacity of locomotives on grades.

26. On a 1.2 per cent grade, base lines of 440 feet each are laid out, the bases being 8,200 feet apart. A train with cars weighing 590 tons, drawn by a consolidation engine weighing 107 tons, runs up the grade, passing the lower base in 19.6 seconds and the upper base in 11.2 seconds. What is the total excess power of the engine? How many additional tons could it draw up the grade if the normal tractive resistance is 6 pounds per ton?

27. How much additional energy per ton must an engine exert in order to give to a train a velocity of 30 miles per hour in a distance of 2,000 feet, starting from rest?

28. Discuss the economics of heavy rails.

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